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INTERACTION OF REINFORCED CONCRETE FRAME-
CRACKED SHEAR WALL SYSTEMS
SUBJECTED TO EARTHQUAKE LOADINGS

by

Gilberto Areiza

A Thesis

Presented to the Graduate Committee

of Lehigh University

in Candidacy for the Degree of

Master of Science

in

Civil Engineering

Lehigh University

Bethlehem, Pennsylvania

July 1979

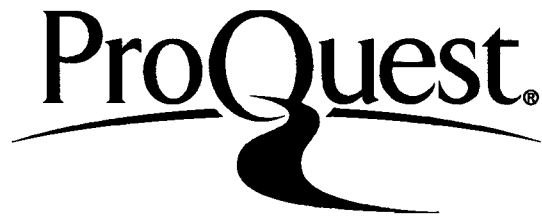
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CERTIFICATE OF APPROVAL

This thesis is accepted and approved in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

July 28, 1979

Dr. Celal N. Kostem
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ACKNOWLEDGMENTS

The research leading to this thesis was conducted at the Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania.

A special debt of gratitude is due to Dr. Celal N. Kostem, Professor of Civil Engineering, who supervised the research and the preparation of the thesis, and whose invaluable suggestions made possible its completion.

The generous support and expert assistance of the staff of the Lehigh University Computing Center permitted the successful completion of the reported research program.

The author expresses his deep appreciation to Mrs. K. Michele Kostem, who revised and corrected several drafts and the final copy of the thesis, and typed the final manuscript.

The Latin American Scholarship Program of American Universities' support is gratefully acknowledged for its support of the author's graduate studies at Lehigh University.

This thesis is dedicated to my wife and my little son.

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ABSTRACT

High-rise reinforced concrete frame structures require special structural arrangements, if they are to be subjected to appreciable lateral loads such as high wind pressures, and especially earthquake loadings. One of the practical methods that has been gaining greater popularity and acceptance is the use of the reinforced concrete shear wall through the height of the building in one or more bays.

The complementary lateral stiffness properties of the frame and the shear wall result in substantial reductions in lateral deflection. The combined frame-shear wall, even though it provides many conveniences, also provides new challenges. The true interaction of the planar frame-shear wall has not been defined even for the static loadings; in the case of earthquake loadings, where the efficiency of the structural system is at its best, the interaction is least understood.

The reported research utilized two different frames stiffened with two different types of shear walls with each wall having five different dimensions, thereby resulting in the analysis of 20 structural systems. The analysis is carried out by using finite element method, and assuming that the structural system will remain linear elastic in the course of the loading. The natural periods of vibration of the structural systems have been accurately computed and comparisons have been provided with the current design codes.

The study has been extended to the structural systems where the shear walls have X-cracking, due to a previous earthquake or primary shock of the earthquake under consideration. The structural and vibrational characteristics of the frame-shear wall system have been recomputed considering the damaged walls. Attempts have been made to correlate the structural degradation in the shear wall, due to the cracking, and the static and dynamic response of the structural system with and without the imposed damage.

Special attention is paid to the behavior of the structural systems when subjected to lateral loadings. The results have been presented in the form of deflection profiles, periods of vibration the total base shear developed, and the percentages of base shear taken by the frame and by the shear wall. Tentative guidelines are provided for the preliminary dimensioning of the shear walls, if they are to be combined with the reinforced concrete frames. The research concluded that (1) for high-rise structural systems frame and shear wall should be designed to have complementary and compatible displacements, (2) in "reasonable" structural systems the frame carries 15% of the base shear, and (3) static equivalent lateral load in seismic design, according to the Uniform Building Code, could be increased 40% to 70%.

1. INTRODUCTION

During the last three decades increased design and construction of high-rise reinforced concrete buildings are noted. The current trend indicates that in the future there will be an increase in the heights of this type of construction. Several factors account for this rapid development of reinforced construction, which may range from economic factors, like the lack of a strong steel industry in certain countries, which makes high-rise steel buildings very expensive as compared to high-rise concrete ones, to aesthetic requirements and architects' personal preferences.

Depending upon the number of stories, several structural systems have been used. Frame structures, which depend entirely on the rigidity of the frame connection for their performance under vertical and lateral loads, have been built up to heights of about 60 stories (Ref. 3). They, nevertheless, tend to be uneconomical beyond 10 or 15 stories due to the additional structural provisions required for lateral loads. In general, for increased heights, structural engineers increase the structural member sizes over those required for vertical loads. This can be referred to as "premium," i.e. increase in cost due to lateral loads.

Since the most efficient multistory structure is that which pays the minimum premium in order to provide the necessary stiffness for lateral loads, structural engineers usually have to use other configurations when dealing with tall concrete buildings. This has

led to the development of structural systems like shear wall, frame-shear wall, framed tube, tube-in-tube and modular tube. A discussion of their advantages and optimization criteria is reported by Derecho, and Khan and Iyengar (Refs. 3,8).

If a deep vertical element or shear wall is subjected to lateral loading, it will deform in a bending mode and its deflected shape is similar to that of a cantilever beam (Fig. 1), whereas the deflection profile of a framed structure is analogous to that of a fixed-ended beam subjected to support settlement (Fig. 2). When these two structural components are put together to form a different structural system, interaction forces, which enforce equal lateral deformations at the floor levels, are developed and an interesting case of indeterminacy is created. The interaction between these two elements is such that the frame tends to reduce the lateral deflection of the shear wall at the top while the wall supports the frame near the base (Fig. 3).

2. FRAME-SHEAR WALL INTERACTION

2.1 Analysis and Design of Frame-Shear Wall Systems

Although frame-shear wall structures have been investigated, designed and built in the past years, little is known about the interaction mechanism due to the complicated nature of the problem. An accurate analysis of these structural systems requires the coupled solution of elasticity formulation for the shear wall and matrix formulation for the frame. This corresponds to a prohibitive proposition for the analysis of all structural systems, except a few extremely simple configurations.

The design process of a frame-shear wall structure has four stages (Ref. 16). The first is the conceptual stage where the different criteria are established, the architectural and planning requirements are met and a tentative decision is made about the location and shape of the shear walls. The second is the analysis of the structural systems: the forces acting on each element are determined. Thirdly the stresses are checked and the required modifications are made to comply with the strength and code requirements. Finally, detailed design computations and plans are completed.

Due to the high degree of indeterminacy of the system, the second stage is usually the time-consuming part in the process. Or conversely, at this stage in order to reduce the computational effort, many dubious assumptions could be introduced, depending

upon the desired simplicity. Prior to the development of computer-oriented techniques, special approximate manual methods were developed and used for many years. The different approaches are summarized by Notch and Kořtem (Ref. 14).

With the development of matrix structural analysis techniques, the increasing availability of computer programs for accurate analysis and the advent of the finite element method, the approximate manual methods of analysis of frame-shear wall systems have gradually become obsolete. Using the finite element method, frame-shear wall structures can be realistically modeled for an analysis scheme of the required accuracy (Ref. 6).

In common with other procedures for numerical solutions in structural engineering problems, the finite element method requires the formation and solution of a large number of linear simultaneous algebraic equations. The special advantage of the method resides in its ability for automation of the equation formation process and in its ability to represent highly irregular and complex structures and loading conditions. Special situations in frame-shear wall systems, like post-cracking behavior, can be easily handled by this method.

2.2 The Scope of the Reported Research

One of the many problems that a structural engineer faces during the design process of a frame-shear wall system is to evaluate the effectiveness of a particular shear wall prior to a detailed

computer analysis. This is due to the scarcity of qualitative and quantitative information on behavior of shear walls, and especially shear wall-frame interaction. The reported research was undertaken to identify trends in the structural behavior of this type of system in order to develop tentative guidelines in dimensioning both frame and shear wall; which may result in savings in final design time and final design costs.

Engineers designing for seismic loads are always concerned about ductility and post-cracking behavior of the frame-shear wall systems (Refs. 9,10). This is due to the fact that the imposed seismic loads may be several times greater than the "allowable static strength" of the shear wall (Ref. 16). Consequently, special attention must be given to the post-cracking behavior of the system in order to incorporate ductility requirements into the design process. Even though the importance of the ductility of the shear wall and post-cracking behavior is recognized by all designers and analysts, very little is known of these phenomena. The last part of this investigation is devoted to this aspect.

Because of the presence of the many variables that will affect the structural response of frame-shear wall systems, an all inclusive investigation is not practical. However, a parametric investigation of limited scope and objectives can still be undertaken to identify the critical design parameters that govern the structural response. Impositions of limitations will inevitably lead to restrictions on the applicability of the findings of the research

program. The final results of the research will be in the form of tentative guidelines to assist designers in better understanding of the structural systems; rather than a set of curves, tables or formulae that can enable the designer to by-pass the required analysis phase. Since design can be considered as a repetitive analysis, the implementations of the findings reported herein can reduce the number of "repetitive analyses." The above discussion is the fundamental philosophy in the definition of the scope and the conduct of the reported research.

Two previously designed reinforced concrete frames are used in the parametric investigation. The frames are "attached" to shear walls of various dimensions. Two different placements of the shear walls with respect to the frame are also investigated. Thus, several shear wall-frame configuration types are analyzed to provide information regarding lateral deflection profiles, base shear distribution and vibrational characteristics.

The investigation is then extended to structural systems with cracked shear walls, to provide quantitative information on the effects of structural deterioration on the response of structural systems.

3. ANALYSIS OF THE FRAME-SHEAR WALL SYSTEMS

3.1 Description of the Frames

One of the frames investigated is a three-bay ten-story frame reported by Zagajeski and Bertero in their research program and described in "Computer-Aided Optimum Seismic Design of Ductile Reinforced-Concrete Moment-Resisting Frames" (Ref. 17). This frame is referred to herein as Frame 1. The dimensions and design loads for this frame are shown in Fig. 4 and member sizes are shown in Fig. 5. It is a rigid concrete frame designed to carry dead and live loads according to the American Concrete Institute Specifications (Ref. 18). The resistance to lateral forces entirely depends upon the rigidity of the member connections.

The second frame used in the investigation is a three-bay twenty-story reinforced concrete frame taken from the report by Clough and Benuska, "FHA Study of Seismic Design Criteria for High-Rise Buildings" (Ref. 2). The frame is referred to herein as Frame 2. The pertinent dimensions and working loads for this frame are shown in Fig. 6 and member sizes are shown in Fig. 7. The building was originally designed to carry vertical loads plus the static lateral forces prescribed by the 1964 Edition of the Uniform Building Code using simple approximate analysis procedures.

3.2 Frame-Shear Wall Configurations

Frames 1 and 2 are linked to five different shear walls in two

different types of configurations (Fig. 8); thereby resulting in twenty different structural systems. In Type A frame-shear wall configuration the beams of the second bay are removed and the shear wall is placed in that position. The columns supporting the second bay beams are also removed and full moment-resisting beam-shear wall connection is considered. In Type B frame-shear wall configuration the shear wall is placed adjacent to the last column line, the concrete columns are removed and full moment-resisting beam-shear wall connection is assumed. This results in a quasi-four-bay structural system (Fig. 8). Since the common practice in reinforced concrete frames is the moment connection, shear connection is not considered practical; therefore, it is not included in this investigation.

3.3 Analysis

Each frame is analyzed for the original frame, and Type A and Type B configurations using the finite element computer program SAP IV (Ref. 1). Each frame-shear wall configuration is analyzed considering five choices of shear wall dimensions:

<u>Frame 1 - Shear Wall</u>	<u>Dimensions (Centimeters)</u>
A	30 x 244
B	30 x 305
C	30 x 366
D	30 x 427
E	30 x 488
<u>Frame 2 - Shear Wall</u>	<u>Dimensions (Centimeters)</u>
A	40 x 366
B	40 x 427
C	40 x 488
D	40 x 549
E	40 x 610

Each frame and frame-shear wall configuration type are analyzed for wind, dead and live loads and earthquake excitation. In the analysis for wind load, dead and live loads are considered and combined using the recommendations of the 1977 Edition of the American Concrete Institute Standards (Ref. 18). For wind load analysis, equivalent horizontal static forces acting at each floor level are computed. The study included the following six load cases:

Case 1: dead load only

Case 2: wind load only

Case 3: dead plus wind load

Case 4: factored dead and wind loads

Case 5: factored dead and live loads

Case 6: factored dead, wind and live loads

In the analysis for earthquake loading (1) static equivalent type loads, and (2) dynamic forces through the use of modal superposition technique are considered.

Equivalent horizontal static forces are determined by using the recommendations of the 1976 Edition of the Uniform Building Code (Ref. 20):

$$V = ZIKCSW$$

where:

V = total lateral force to be resisted

Z = numerical coefficient depending upon the seismic zone

I = occupancy importance factor

K = horizontal force factor depending upon the type of

structure

W = total dead load of the structure

C = numerical coefficient based on the natural period of vibration of the structure

$$C = \frac{1}{15\sqrt{T}}$$

T = natural period of vibration in seconds

$$T = \frac{0.05 h}{\sqrt{D}}$$

h = height of the building above base level in feet

D = dimension of the structure in the direction parallel to the applied forces, in feet; or

$T = 0.10 N$

N = total number of stories above base level, when the lateral force resisting system consists of a ductile moment-resisting frame

The total lateral force, V , is distributed over the entire height of the structure according to:

$$V = F_t + \sum F_i$$

The concentrated force at the top, F_t , is computed according to:

$$F_t = 0.07(TV) \leq 0.25 V$$

The remaining portion of the total base shear, V , is distributed over the entire height of the structure including the top level according to:

$$F_i = \frac{(V - F_t) w_i h_i}{\sum w_i h_i}$$

where:

w_i = weight of the i th level

h_i = height above the base level to the i th level

In the phase of analyses that included equivalent static earthquake loads, four load cases were developed using ACI Standards (Ref. 18):

Case 1: static earthquake loads only

Case 2: dead plus static earthquake loads

Case 3: factored dead, live and static earthquake loads

Case 4: factored dead and static earthquake loads

The actual dynamic response of the structures is determined by the modal superposition method employing the first five predominant modes, and subjecting the frames and frame-shear wall systems to El Centro Earthquake of May 1940. Ground motion is inputted by response spectrum. The extreme response of the structural system is computed by modal participation factors and square root of the sum of squares approach. Natural periods of vibration are determined as a by-product in the process.

3.4 Frame-Cracked Shear Wall Systems

The last part of the investigation is devoted to the study of the post-cracked behavior of the frame-shear wall systems.

During an earthquake excitation, strong horizontal

accelerations result on the building masses producing horizontal loads. On the other hand, vertical dead and live loads act on each story of the structure. Therefore, each shear wall panel is subjected to vertical and lateral loads, and the panel is in bi-axial state of stress. The principal stresses in the wall will be in the direction of the diagonals through most of the height of the shear wall. However, near the base of the shear wall due to the transfer of the base shear, depending upon the overall structural configuration, the wall may be subjected to a different mode of stress. It may either be in essentially flexural or essentially shear mode, or a combination thereof. The last mode would be closer to the previously stated state of stress, i.e. principal stresses being in the direction of the diagonals.

3.4.1 Damage Mechanism

Shear wall or any similar units that are built to perform like a diaphragm are the stiffest components of the overall frame-shear wall system. Consequently, these walls tend to carry the larger percentage of the lateral loads. This continues to be the case until shear wall develops local structural degradation and loses part of its lateral stiffness. It is shown by Kostem and Green that masonry infill walls bounded by the reinforced concrete frame increase the lateral stiffness of the structure, even though the masonry was not "intended" to perform as a lateral stiffening unit (Ref. 11).

At increased load levels the infill walls will exhibit the first sign of distress. It can be concluded that in structural frame-shear wall systems the walls are more susceptible to damage than the rest of the structure (Refs. 9,10).

In the case of reinforced concrete frames with shear walls subjected to large seismic loadings, the wall base may sustain the first damage. This is due to the large amount of base shear that is directly transmitted to the structure through the shear wall (Ref. 16). However, if the wall is designed properly, with sufficient attention paid to the lower levels of the wall, than the possible failure of the wall near the support can be prevented, or at least retarded.

Field observations and analytical studies of the earthquake damage to the frame-shear wall structural systems have clearly indicated that the primary mode of damage sustained by the shear walls is the formation of X-cracks or diagonal cracks (Refs. 9,10). These cracks occur at shear wall panels defined by the vertical boundaries of the shear wall and the consecutive beam axes. The cracks in the panel will extend from lower left to upper right corner, and similarly from lower right to upper left corner. Due to the structural imperfections, and especially due to the build up of seismic forces differently at different floors, the diagonal cracks do not necessarily occur at each floor, but randomly through the height of the shear wall.

3.4.2 Structural Idealization and Soft Story Concept

One of the major difficulties in analyzing the shear wall-frame system is to "extract" the planar unit out of a truly three dimensional structural configuration. The research by Fintel and Gosh have provided examples for this process (Ref. 5). After isolation of the planar structural system, its analysis, either in elastic regime or in inelastic regime, becomes manageable. However, additional research by Kostem and Heckman have indicated that the state-of-the-art in the isolation of the planar structural system from a three dimensional structure has not progressed sufficiently (Refs. 12,13). This is primarily due to the contribution of the floor system to the lateral stiffness of the structure and the torsional effects that may exist in the actual structure. Since the accurate identification of the planar frame-shear wall system may require substantial engineering judgment and/or dubious assumptions, depending upon the actual building configuration, in the reported research no attempt has been made to relate the investigated planar structural systems to actual three dimensional ones.

The recent approach by many researchers has been the use of the soft story concept. This assumes that the lower levels of the shear wall will loose its inherent stiffness in the course of the earthquake. This assumption, coupled with the gross reduction of the actual two dimensional frame-shear wall

combination into a much simpler one, permits the use of the time-history analysis of the structural system, which will also permit the inclusion of various forms of nonlinearities, hysteresis loops, etc (Refs. 4,5,7). In the design of the reported research this approach has not been considered. This is due to the fact that time history analysis is an accurate, but laborious approach; however, if the reduction of the two dimensional structural system into a much simpler one can not be accurately done, than the results may contain large errors.

3.4.3 Assumed Damage Mechanism

A different analytical modeling is employed in place of soft story concept, reduction of the structural system and the time history analysis. The planar structural system is analyzed without any reduction in number of members or joints, i.e. full scale analysis of the combined shear wall and reinforced concrete frame. This permits the results of the computer based analysis to be identified on a one-to-one basis with the actual structural components.

Rather than employing an accurate, but extremely expensive analysis scheme which will start with the intact structure and will progressively identify the damaged regions in the course of the earthquake, i.e. time history analysis, a different but substantially less expensive approach has been

taken. Different amounts of structural degradation are imposed on the shear wall, than the structural system is analyzed for static and/or dynamic loads. By changing the amount of imposed damage, it is then possible to simulate the structural systems with various degrees of structural degradation.

It was observed that the structural frame sustains very small amounts of damage, if any, while the shear wall is exhibiting some form of cracking. Therefore, in the research it is assumed that the beams and columns remain linearly elastic. Thus, in the phase of the research dealing with the cracked shear walls only, the type of damage that is considered is the diagonal cracks in the walls at each floor level. In actual structural damage the cracked shear wall panels do not necessarily happen at each and every floor, in the research, for the sake of simplicity, it is assumed that shear wall exhibits the same type of damage at each floor level (Fig. 24). This eliminates one of the major obstacles in the parametric investigation, which is the variation of the amount of damage and extent of spread in the shear wall.

3.5 Mechanical Properties

The concrete for beams in both frames is assumed to have a 28 day cylinder compressive strength of 20.685 MPa, while the compressive strength of the concrete for columns and shear walls is assumed to be 27.58 MPa. The modulus of elasticity for beams is 21.53 GPa

and for columns and shear walls is 24.86 GPa. Poisson's ratio for the concrete is taken as 0.15.

3.6 Analytical Modeling

The static and dynamic response of the structural system is simulated using the finite element displacement method and program SAP IV (Refs. 1,6). The dynamic analysis is carried out using the modal superposition technique.

3.6.1 Modeling Assumptions

It is assumed that the structural system is fully linear elastic both for static and earthquake loadings. It is further assumed that:

- * The structural system is planar and remains planar during the loading, which is in the plane of the structural system.
- * Beams and columns can be simulated by beam-column elements, having flexural, shear and axial deformation capabilities.
- * Shear wall is monolithic and can be described by plane stress elements.
- * All beam to column and beam to shear wall connections are rigid connections, i.e. moment connections.
- * Column to foundation as well as shear wall to foundation connections are rigid, i.e. non-yielding supports.

- * The contribution of the floor stiffnesses is neglected.
- * Secondary effects, such as P- Δ effects, are not included.

3.6.2 Modeling of Cracked Walls

The cracked shear walls are simulated by modifying the elastic properties of the appropriate plane stress finite elements (shear wall) in the appropriate directions. Plane stress elements in the assumed cracked regions are modeled as anisotropic. The modulus of elasticity perpendicular to the assumed crack direction is reduced by a predetermined amount. The modulus of elasticity in the direction parallel to the cracks is assumed to remain constant. The average shear modulus is computed using the formulae for anisotropic materials (Ref. 6). The Poisson's ratio is kept constant for cracked and uncracked walls.

By changing the modulus of elasticity in the direction perpendicular to the cracking, a different amount of stiffness degradation is approximated. Slightly damaged shear walls can be simulated with a slight reduction in the modulus of elasticity; whereas severely damaged walls will require substantial reduction in the modulus of elasticity. The results presented in Chapter 4 are based on slight-to-moderately damaged shear walls.

3.6.3 Piecewise Linearization

The correct analytical simulation of the structural system requires the use of numerous beam-column elements and plane stress elements, as well as input of the time history of the ground motion. The equations of motion, one second order differential equation per degree of freedom, need to be solved for each increment of time. The stresses at the members can then be computed, and the elastic properties will be modified, using the proper nonlinear stress-strain relationship and failure criteria, if need be. A formulation as such would yield a continuous "smooth" nonlinear response curve for the structural system. However, this scheme requires extremely large computational efforts, so much so that it would not permit the execution of a parametric investigation.

The reported research employs a piecewise linearization of the inelastic response of the structural system. Rather than determining the level of degradation in the elastic properties of the shear wall, depending upon the state of stress, the elastic properties of a given region are pre-assigned simulating the possible damage that the shear wall would have exhibited. Therefore, the obtained response curve will not be a "smooth" continuous curve, but a combination of straight line segments within the vicinity of the actual curve.

The accuracy of the reported approach could be increased,

depending upon the availability of the computer resources, by altering the preassigned damage patterns a small amount from one configuration to another. However, it should be realized that the attainment of the exact "smooth" response curve could not be accomplished by this approach unless the analyst is familiar with the location of the initiation of damage, and its spread pattern. This is a nearly impossible requirement, especially if the structural system is not a trivially simple one.

4. RESULTS

4.1 General Comments

The primary interest of this investigation is to identify trends for reinforced concrete frame-shear wall systems in order to provide means of assessing the effectiveness of a particular shear wall prior to a more refined analysis or redimensioning. Although dead and live loads are considered in the analysis, the information reported herein related only to lateral loads. It is assumed that the primary function of the shear wall is to provide the necessary stiffness to resist lateral loads, even though the optimum design is one which makes total use of the shear wall to carry lateral and vertical loads (Ref. 14). Therefore, the main emphasis of the results presented is in regard to the behavior of the structural system when subjected to lateral loads. The reported research resulted in a massive amount of information, as most finite element method based investigations do; however, for the sake of brevity the emphasis in the presentation of the results is placed on deflection profiles. Special attention is devoted to the study of post-cracked characteristics of the structural systems. Specifically, the information presented in this report corresponds to:

1. Deflection profiles for selected frame-shear wall configurations.
2. Percentages of base shear taken by the frame and by the shear wall for chosen combinations of frame-shear wall configurations.

3. Natural periods of vibration and dynamic characteristics of the structural systems.
4. Post-cracked wall behavior of the system related to:
 - a. Deflection profiles and top deflection increments.
 - b. Changes in distribution of base shear.
 - c. Increments in natural periods of vibration.

4.2 Deflection Profiles

Even though there are several parameters which can be used to "measure" the interaction between frames and shear walls, the one frequently used is the deflection profile because it represents the best index to show the effectiveness of a shear wall on a frame system and vice versa. Figs. 1 and 2 show deflection profiles for isolated frame and shear wall respectively. Fig. 3 shows the deflection profile for the combined structural system and, as it can be observed, the deflected shape is quite different from the first two, and the deflection index measured as the lateral displacement at the top is smaller than in the first two cases. The effectiveness of frame-shear wall interaction can be best illustrated by the following example. The Marina City tower is the first known building in which the lateral load was assigned to the frame and to the central core resulting in a top lateral displacement of 100 millimeters. An initial analysis was performed assigning the entire lateral load only to the shear wall resulting in a top lateral displacement of 400 millimeters.

Deflection profiles for Frame-1-Shear Wall and for Frame 2-Shear Wall configurations are plotted in Figs. 9-13 and 14-18 respectively. The deflection profiles for each frame and shear wall alone are included in each figure to illustrate the deformation mode for each structure and to provide bases to evaluate the effect of one of the structures on the other. A total of four displacement patterns is shown in each figure:

- + - Frame alone
- Δ - Shear wall alone
- * - Frame-Shear Wall System - Type A configuration
- ☆ - Frame-Shear Wall System - Type B configuration

It can be noted that there exists a similarity between the deflection profiles for Type A and Type B configurations. It is important to note that the differences in floor displacements and top deflections between Type A and Type B configurations become smaller as the shear wall length increases. In all cases Type B configuration produces the stiffest frame-shear wall combination. This is due to (1) increase in the total horizontal length (i.e. "D") of the structural system, and (2) placement of the shear wall at the extremity of the structure, rather than the "core."

Values ranging from 1/300 to 1/600 have been used in practice as drift limits due to wind loads, depending upon the judgment of the engineer (Ref. 3). The higher value appears to be more appropriate for the traditional building types of several decades ago where so-called "non-structural" heavy masonry walls increased

considerably the lateral stiffness of frames. With the actual trends of using lightweight elements as partitions and walls a relatively smaller value has been used. A reasonable value of about $1/400$ yields results of 94.5 millimeters for Frame 1 and 185.2 millimeters for Frame 2. Top deflection varies from 41.1 to 17.8 millimeters for Frame 1-Shear Wall Type A configuration and from 17.1 to 11.1 millimeters for Frame 1-Shear Wall Type B configuration as the shear wall length increases. For Frame 2-Shear Wall configurations, top deflection varies from 196.1 to 93.4 millimeters and from 99.6 to 65.8 millimeters for Type A and Type B configurations respectively.

For all choices of the shear wall dimensions on Frame 1, top deflections are well within the drift limit. The top deflection of Frame 1 alone is also within this limit. This indicates that this frame, as originally designed, is rigid enough to support lateral loads and that wind loads have very little effect on it. Frame 2, however, is more susceptible to wind effect. For the shorter shear wall length Type A configuration, the top deflection exceeded the sway index by 6%, although the top lateral displacement for Frame 2 alone is within the drift limit.

4.3 Distribution of Base Shear

The total horizontal forces at the base, taken by the frame and by the shear wall, are extracted from the computer outputs and are shown in Table 1 and Table 2 for Frame 1-Shear Wall configurations and for Frame 2-Shear Wall configurations

respectively. Percentages of base shear as a fraction of the total lateral force applied are determined and are also shown in Tables 1 and 2. A graphic representation of the percentages of base shear on Frame 1 and on Frame 2 is plotted in Figs. 19 and 20 respectively, for the different configurations and for the different shear wall lengths.

Percentages of base shear on shear wall for Frame 1-Shear Wall configurations, shown in Table 1, range from 75% to 92% for Type A configuration and from 52% to 79% for Type B configuration. These values indicate how stiff this frame is as originally designed and the relatively small effect of the shear wall on this frame. This conclusion could be expected since this building is relatively short and can be designed relying upon the rigidity of the frame connections to carry lateral loads.

On the other hand, percentage of base shear on shear wall for Frame 2-Shear Wall configurations, shown in Table 2, ranges from 91% to 96% for Type A configuration and from 78% to 89% for Type B configuration, which indicates the effectiveness of the shear wall on this frame.

The graphic representation of the percentages of base shear taken by the frames, shown in Figs. 19 and 20, indicates that Type A and Type B configurations produce approximately the same distribution. However, the percentage of base shear taken by the frame part in Type B configuration is larger than the percentage of base

shear taken by the frame part in Type A configuration due to the effect of the third column line and the second beam bay, which are not included in the latter configuration. Finally, these two figures also show that the difference in base shear taken by the frame part of the frame-shear wall system becomes smaller as the shear wall length increases, which is reasonable because the shear wall is more effective as its length increases.

4.4 Seismic Considerations

Natural periods of vibration for the frames and for the frame-shear wall configurations are determined using the finite element program SAP IV and the Uniform Building Code (UBC) recommendations (Ref. 20 - see Section 3). The values obtained are shown in Table 3 for Frame 1 and in Table 4 for Frame 2. The graphic representation of these values appears in Fig. 21 for Frame 1-Shear Wall configurations and in Fig. 22 for Frame 2-Shear Wall configurations. The word "STATIC" in both figures stands for the natural periods of vibration as determined by the UBC recommendations, although it is not the most appropriate name.

"C" factors, used to compute the total equivalent lateral force ($V = ZIKCSW$) for earthquake analysis, are computed based on T values from finite element analysis (SAP IV) and on T values from UBC formulae; and are also presented in Tables 3 and 4. The variation in T between finite element analysis (SAP IV) and UBC formulae ranges from 40% to 65%, while the percent variation for

the "C" factor ranges from 30% to 70%. It can be noted that natural periods of vibration from UBC recommendations are smaller than the values obtained by finite element analysis (SAP IV), which means that UBC recommendations consider stiffer structures which take more earthquake loads. For the design of frame-shear wall systems to resist earthquake loads using UBC recommendations, the structure has to withstand from 1.4 to 1.7 times the equivalent static load if the natural period of vibration from UBC formulae is used.

From the graphic representation it can be observed that the variation, as well as the actual periods of vibration themselves for Type A and Type B configurations, decrease with increasing shear wall length. The periods of vibration asymptotically approach zero seconds as the stiffness of the structure approaches infinity.

It is possible that the natural period of vibration of the actual structure will be less than the value obtained by the analysis due to stiffening non-structural elements such as partitions, walls, elevator shafts and stairs. However, these secondary structural components are not explicitly contained in the UBC recommendations and the comparison of natural periods of vibration carried out in this investigation is still valid.

Comparison of fundamental periods of the frame-shear wall configurations determined by the finite element analysis and by the approximate formula of the Applied Technology Council is presented in Appendix-A (Ref. 19).

4.5 Post-Cracked Behavior

4.5.1 Deflection Profiles

Deflection profiles for Frame 1-Cracked Shear Wall and for Frame 2-Cracked Shear Wall configurations are shown in Figs. 25-29/and Figs. 30-34 respectively. Deflection profiles for the uncracked frame-shear wall configurations are plotted in the same figures to provide a basis for comparison. A total of four displacement patterns is plotted in each figure. In order to distinguish the deflection patterns, different symbols are used for the configuration types and for the shear wall conditions: uncracked or cracked. The symbols used are:

+ - Frame-Uncracked Shear Wall - Type A Configuration

Δ - Frame-Cracked Shear Wall - Type A Configuration

* - Frame-Uncracked Shear Wall - Type B Configuration

☆ - Frame-Cracked Shear Wall - Type B Configuration

In addition, at the bottom of each figure there is a label which identifies the particular shear wall whose results are shown in the plot. For instance, for Fig. 30 the label identifies the plot for the specific combination Frame 2-Cracked Shear Wall A, whose shear wall length is 367 centimeters.

The effect of the cracked wall on the deflection profiles cannot be observed easily for shorter shear wall lengths in the Frame 1-Shear Wall configurations, because of the relatively little importance of the shear wall in the overall

behavior of the system for this particular case. As the shear wall length increases the effect of the cracking becomes more important and the deflection profiles present an appreciable lateral displacement increment.

The effect of the cracked wall in the Frame 2-Shear Wall configurations is relatively small, although noticeable enough, for shorter shear wall lengths. It also presents the same tendency of becoming more important as the shear wall length increases.

Increases in top deflection are determined for Frame 1-Cracked Shear Wall and for the Frame 2-Cracked Shear Wall configurations and the values are reported in Tables 5 and 6, and graphically in Figs. 35 and 36 respectively. Top deflections incremented from 0.01% to 27% for Frame 1-Cracked Shear Wall configurations and from 3.5% to 41% for Frame 2-Cracked Shear Wall configurations. For both cases the increment is larger for Type A configuration since the shear wall is more important in this case. The variations of top deflection between Type A and Type B configurations, as well as the deflections themselves, increase as the shear wall length increases.

4.5.2 Distribution of Base Shear

The total reaction lateral forces acting on frame and on shear wall are determined by applying the same procedure used before for the uncracked shear wall-frame configurations.

Percentages of base shear are determined as a fraction of the total base shear and the results are presented in Tables 7 and 8 for Frame 1-Cracked Shear Wall and for Frame 2-Cracked Shear Wall configurations respectively. Plots of the percentages of base shear taken by the frame part of the frame-cracked shear wall systems are presented in Figs. 37 and 38.

Percentages of base shear on shear wall range from 52% to 92% for Frame 1-Cracked Shear Wall configurations and from 78% to 95% for Frame 2-Cracked Shear Wall configurations.

Percentage increments of base shear acting on frame, for the different frame-cracked shear wall configurations, are determined and presented in Tables 9 and 10 for Frame 1-Cracked Shear Wall and for Frame 2-Cracked Shear Wall configurations respectively. The values presented in these tables are shown graphically in Figs. 39 and 40 respectively. These values range from 0.70% to 13% for Frame 1-Cracked Shear Wall configurations and from 0.50% to 22% for Frame 2-Cracked Shear Wall configurations. In both cases the increment of base shear is larger for Type B configuration, which is a reasonable result because of the more relevant effect of the frame in this configuration type.

4.5.3 Seismic Characteristics

Post-cracked shear wall effects on the dynamic characteristics of the frame-shear wall configurations are considered in

the investigation process. Natural periods of vibration under these circumstances are determined using the finite element program SAP IV and the results are reported in Figs. 41 and 42 for Frame 1-Cracked Shear Wall and for Frame 2-Cracked Shear Wall configurations respectively. A tendency similar to the one exhibited by the frame-uncracked shear wall configurations is observed. The variations in natural periods of vibration, as well as the actual periods of vibration themselves for Type A and Type B configurations, decrease with increasing shear wall length.

Percentage increments in natural period of vibration are determined and reported in Tables 11 and 12 and plotted in Figs. 43 and 44 for Frame 1-Cracked Shear Wall and for Frame 2-Cracked Shear Wall configurations respectively.

Larger effects, as expected, are reported in Type A configuration for both cases due to the larger contribution to the stiffness of the overall system done by the shear wall in this case. Natural period of vibration increments range from a very small value to 14% for Frame 1-Cracked Shear Wall and from 2% to 20% for Frame 2-Cracked Shear Wall configurations. Also as expected, natural periods report larger increments for Frame 2-Cracked Shear Wall configurations. The variation in natural periods of vibration increments, as well as the increments themselves, increase with increasing shear wall length.

5. CONCLUSIONS

In order to provide guidelines for assessing the effectiveness of a particular shear wall on a reinforced concrete frame, two previously designed frames were linked to five shear walls in two different configuration types. The following conclusions may be drawn from this research:

1. The type of frame-shear wall configuration has less and less effect on the lateral displacements as the shear wall length increases.
2. Special attention must be given to the design of frame-shear wall systems to match sway requirements as the height of the structure increases.
3. The percentage of base shear taken by the frame was approximately 15% for "reasonable" choices of shear wall dimensions and frame member sizes.
4. The differences in natural periods of vibration between Type A and Type B configurations become smaller as the shear wall length increases.
5. Special attention must be given to the design of frame-shear wall systems to support earthquake loads when using UBC recommendations. The structure has to withstand from 1.4 to 1.7 times the equivalent static load if the T value from UBC formulae is used.
6. Ductility provisions are to be established to assure safe post-cracked behavior of the frame-shear wall systems.

Lateral displacement increments ranged from 3% to 40%.

Percentage increments of base shear taken by the frames ranged from 1% to 22%. Increments in natural periods of vibration were reported up to 20%.

7. Additional parametric studies should be conducted on frame-undamaged-shear wall combinations of different geometries to verify the quantitative findings of the reported research.
8. Additional parametric studies should be conducted for the investigated frames with damage of different magnitude.
9. Additional parametric studies referred to in conclusion No. 7 should be extended to damaged configurations parallel to conclusion No. 8.

TABLES

TABLE 1: DISTRIBUTION OF BASE SHEAR

FRAME 1 - SHEAR WALL CONFIGURATIONS

Shear Wall- Frame Configuration	Shear Wall Length* (Centimeters)	Base Shear (Newtons)		Percentage of Base Shear	
		On Frame	On Shear Wall	On Frame	On Shear Wall
Type A	243.84	85801.92	262476.48	24.70	75.30
	304.80	59425.28	288853.12	17.10	82.90
	365.76	43723.84	304554.56	12.60	87.40
	426.72	33715.84	314562.56	9.40	90.60
	487.68	26999.36	321279.04	7.80	92.20
Type B	243.84	167783.01	180495.39	48.20	51.80
	304.80	130904.64	217373.76	37.60	62.40
	365.76	104643.65	243634.75	30.10	69.90
	426.72	85784.13	262494.27	24.60	75.40
	487.68	71933.06	276345.34	20.70	79.30

*The shear wall width remained constant: width = 30.48 centimeters.

TABLE 2: DISTRIBUTION OF BASE SHEAR
FRAME 2 - SHEAR WALL CONFIGURATIONS

Shear Wall - Frame Configuration	Shear Wall Length* (Centimeters)	Base Shear (Newtons)		Percentage of Base Shear	
		On Frame	On Shear Wall	On Frame	On Shear Wall
Type A	365.76	57512.64	597188.48	8.80	91.20
	426.72	44702.40	609998.72	6.80	93.20
	487.68	35939.84	618761.28	5.50	94.50
	548.64	29623.68	625077.44	4.50	95.50
	609.60	24908.80	629792.32	3.80	96.20
Type B	365.76	142291.52	512409.60	21.70	78.30
	426.72	115247.68	539453.44	17.60	82.40
	487.68	95543.04	559158.08	14.60	85.40
	548.64	80686.72	574014.40	12.30	87.70
	609.60	69255.36	585445.76	10.60	89.40

*The shear wall width remained constant: width = 40.64 centimeters.

TABLE 3: NATURAL PERIODS OF VIBRATION

Shear Wall- Frame Configuration	S. W. Length* (Cms)	Period SAP IV (Sec)	Period UBC (Sec)	Percent Variation (%)	"C" Factor Based on SAP IV	"C" Factor Based on UBC	Percent Variation (%)
Frame 1		1.702	1.000	41.25	0.051	0.067	31.37
Type A	243.84	2.270	0.814	64.14	0.044	0.074	68.18
	304.80	2.078	0.800	61.50	0.046	0.075	63.04
	365.76	1.893	0.787	58.43	0.048	0.075	56.25
	426.72	1.720	0.775	54.94	0.051	0.076	49.02
Type B	487.68	1.562	0.763	51.15	0.053	0.076	43.40
	243.84	1.832	0.661	63.92	0.049	0.082	67.35
	304.80	1.759	0.653	62.88	0.050	0.082	64.00
	365.76	1.679	0.646	61.52	0.051	0.083	62.75
	426.72	1.594	0.639	59.91	0.053	0.083	56.60
	487.68	1.506	0.633	57.99	0.054	0.084	54.95

*The shear wall width remained constant: width = 30.48 centimeters.

TABLE 4: NATURAL PERIODS OF VIBRATION

Shear Wall- Frame Configuration	S. W. Length* (Cms)	Period SAP IV (Sec)	Period UBC (Sec)	Percent Variation (%)	"C" Factor Based on SAP IV	"C" Factor Based on UBC	Percent Variation (%)
Frame 2		3.416	2.000	41.45	0.036	0.047	30.56
Type A	365.76	4.263	1.685	60.47	0.032	0.051	59.38
	426.72	3.962	1.653	58.28	0.033	0.052	57.58
	487.68	3.674	1.624	55.80	0.035	0.052	48.57
	548.64	3.406	1.595	53.17	0.036	0.053	47.22
	609.60	3.158	1.569	50.32	0.038	0.053	39.47
Type B	365.76	3.606	1.432	60.29	0.035	0.056	60.00
	426.72	3.486	1.412	59.50	0.036	0.056	55.56
	487.68	3.352	1.394	58.41	0.036	0.056	55.56
	548.64	3.208	1.376	57.11	0.037	0.057	54.05
	609.60	3.058	1.358	55.59	0.038	0.057	50.00

*The shear wall width remained constant: width = 40.64 centimeters.

TABLE 5: TOP DEFLECTION INCREMENT
FRAME 1 - SHEAR WALL CONFIGURATIONS

Shear Wall - Frame Configuration	Shear Wall Length* (Centimeters)	Top Deflection (Millimeters)		Percentage Increment Top Deflection
		Cracked	Uncracked	
Type A	243.84	41.128	41.125	0.01
	304.80	34.839	33.881	2.83
	365.76	29.883	27.529	8.55
	426.72	25.926	22.175	14.14
	487.68	22.665	17.810	27.26
Type B	243.84	17.058	17.056	0.01
	304.80	15.809	15.639	1.09
	365.76	14.713	14.144	4.02
	426.72	13.732	12.621	8.80
	487.68	12.828	11.125	15.31

*The shear wall width remained constant: width = 30.48 centimeters.

TABLE 6: TOP DEFLECTION INCREMENT
FRAME 2 - SHEAR WALL CONFIGURATIONS

Shear Wall - Frame Configuration	Shear Wall Length* (Centimeters)	Top Deflection (Millimeters)		Percentage Increment Top Deflection
		Cracked	Uncracked	
Type A	365.76	210.134	196.096	7.16
	426.72	186.060	163.721	13.96
	487.68	165.415	135.928	21.69
	548.64	147.406	112.629	30.88
	609.60	131.557	93.106	40.84
Type B	365.76	103.101	99.626	3.49
	426.72	97.940	91.366	7.20
	487.68	92.906	82.761	12.26
	548.64	87.912	74.143	18.57
	609.60	82.934	65.822	26.00

*The shear wall width remained constant: width = 40.64 centimeters.

TABLE 7: DISTRIBUTION OF BASE SHEAR

FRAME 1 - CRACKED SHEAR WALL CONFIGURATIONS

Shear Wall - Frame Configuration	Shear Wall Length* (Centimeters)	Base Shear (Newtons)		Percentage of Base Shear	
		On Frame	On Shear Wall	On Frame	On Shear Wall
Type A	243.84	86427.38	261851.02	24.80	75.20
	304.80	60123.43	288154.97	17.20	82.80
	365.76	44438.07	303840.33	12.80	87.20
	426.72	34756.67	313521.73	10.00	90.00
	487.68	29210.02	319068.38	8.40	91.60
Type B	243.84	167938.67	180339.73	48.40	51.70
	304.80	131327.20	216951.20	37.70	62.30
	365.76	108277.66	240000.74	31.10	68.90
	426.72	92478.37	255800.03	26.60	73.40
	487.68	81131.52	267146.88	23.30	76.70

*The shear wall width remained constant: width = 30.48 centimeters.

TABLE 8: DISTRIBUTION OF BASE SHEAR

FRAME 2 - CRACKED SHEAR WALL CONFIGURATIONS

Shear Wall - Frame Configuration	Shear Wall Length* (Centimeters)	Base Shear (Newtons)		Percentage of Base Shear	
		On Frame	On Shear Wall	On Frame	On Shear Wall
Type A	365.76	57829.97	596860.03	8.90	91.10
	426.72	46027.90	608662.10	7.00	93.00
	487.68	39004.51	615685.49	6.00	94.00
	548.64	43040.54	620649.46	5.20	94.80
	609.60	30322.02	624367.98	4.60	95.40
Type B	365.76	146130.14	508559.86	22.30	77.70
	426.72	123053.92	531636.08	18.80	81.20
	487.68	106707.52	547982.48	16.30	83.70
	548.64	94350.98	560339.02	14.40	85.60
	609.60	84592.06	570097.94	12.90	87.10

*The shear wall width remained constant: width = 40.64 centimeters.

TABLE 9: BASE SHEAR INCREMENT

FRAME 1 - SHEAR WALL CONFIGURATIONS

Shear Wall- Frame Configuration	Shear Wall Length* (Centimeters)	Base Shear on Frame (Nwt.)		Percentage Increment Base Shear
		Cracked Wall	Uncracked Wall	
Type A	243.84	86427.38	85801.92	0.73
	304.80	60123.43	59425.28	1.17
	365.76	44438.07	43723.84	1.63
	426.72	34756.67	33715.84	3.09
	487.68	29210.02	26999.36	8.19
Type B	243.84	167938.67	167783.01	0.09
	304.80	131327.20	130904.64	0.32
	365.76	108277.66	104643.65	3.47
	426.72	92478.37	85784.13	7.80
	487.68	81131.52	71933.06	12.79

*The shear wall width remained constant: width = 30.48 centimeters.

TABLE 10: BASE SHEAR INCREMENT
FRAME 2 - SHEAR WALL CONFIGURATIONS

Shear Wall- Frame Configuration	Shear Wall Length* (Centimeters)	Base Shear on Frame (Nwt.)		Percentage Increment Base Shear
		Cracked Wall	Uncracked Wall	
Type A	365.76	57829.07	57512.64	0.55
	426.72	46027.90	44702.40	2.97
	487.68	39904.51	35939.84	8.53
	548.64	34040.54	29623.68	14.91
	609.60	30322.02	24908.80	21.73
Type B	365.76	146130.14	142291.52	2.70
	426.72	123053.92	115247.68	6.77
	487.68	106707.52	95543.04	11.69
	548.64	94350.98	80686.72	16.93
	609.60	84592.06	69255.36	22.15

*The shear wall width remained constant: width = 40.64 centimeters.

TABLE 11: NATURAL PERIOD OF VIBRATION INCREMENT

FRAME 1 - SHEAR WALL CONFIGURATIONS

Shear Wall- Frame Configuration	Shear Wall Length* (Centimeters)	Natural Period (Seconds)		Percentage Increment (%)
		Cracked	Uncracked	
Type A	243.84	2.273	2.270	0.13
	304.80	2.115	2.078	1.78
	365.76	1.983	1.893	4.75
	426.72	1.872	1.720	8.84
	487.68	1.775	1.562	13.64
Type B	243.84	1.832	1.832	-0-
	304.80	1.775	1.759	0.91
	365.76	1.723	1.679	2.62
	426.72	1.676	1.594	5.14
	487.68	1.632	1.506	8.37

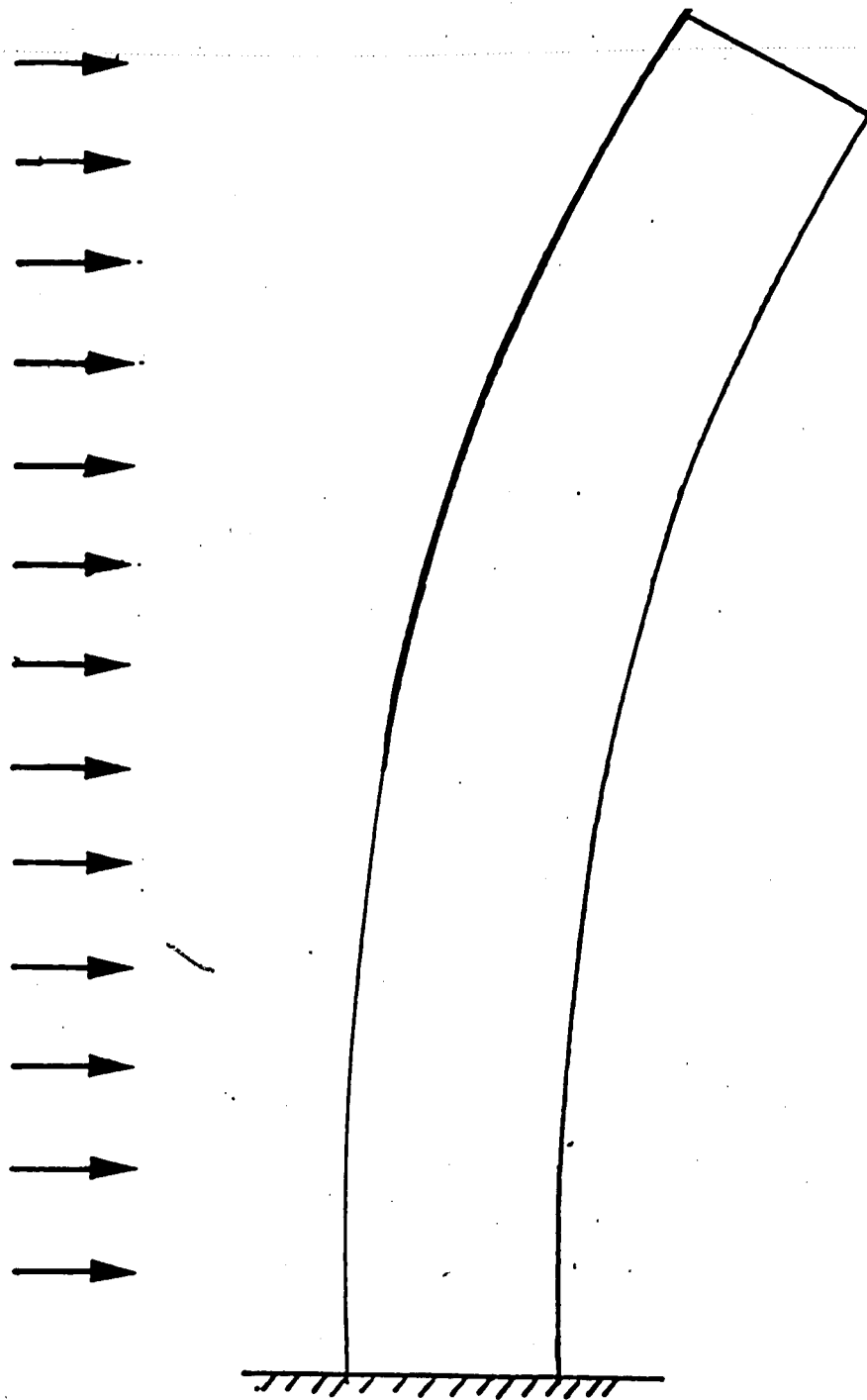
*The shear wall width remained constant: width = 30.48 centimeters.

TABLE 12: NATURAL PERIOD OF VIBRATION INCREMENT
FRAME 2 - SHEAR WALL CONFIGURATIONS

Shear Wall- Frame Configuration	Shear Wall Length* (Centimeters)	Natural Period (Seconds)		Percentage Increment (%)
		Cracked	Uncracked	
Type A	365.76	4.442	4.263	4.20
	426.72	4.259	3.962	7.50
	487.68	4.091	3.674	11.35
	548.64	3.933	3.406	15.47
	609.60	3.783	3.158	19.79
Type B	365.76	3.692	3.606	2.38
	426.72	3.642	3.486	4.48
	487.68	3.591	3.352	7.13
	548.64	3.535	3.208	10.19
	609.60	3.475	3.058	13.64

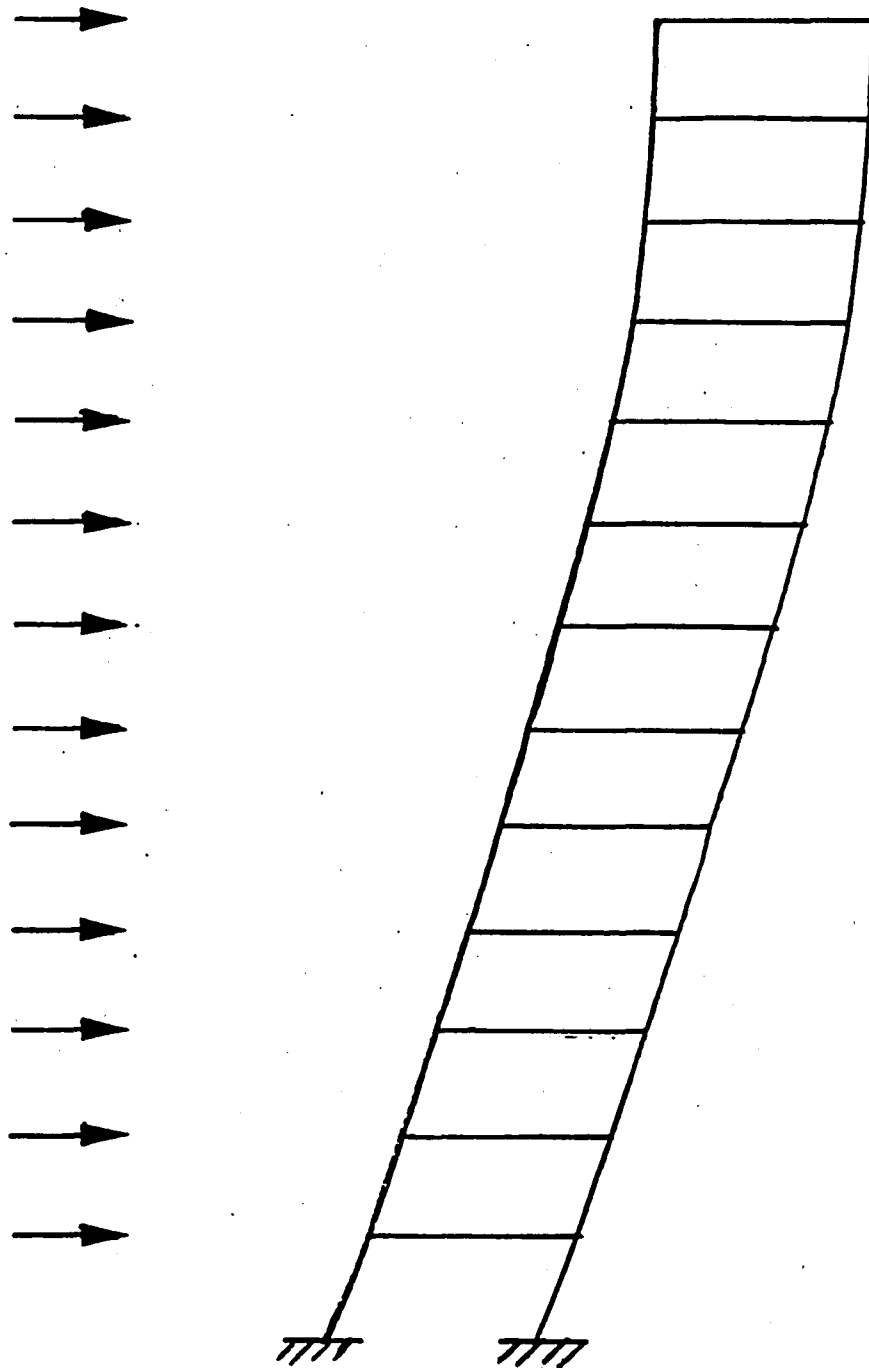
*The shear wall width remained constant: width = 40.64 centimeters.

FIGURES

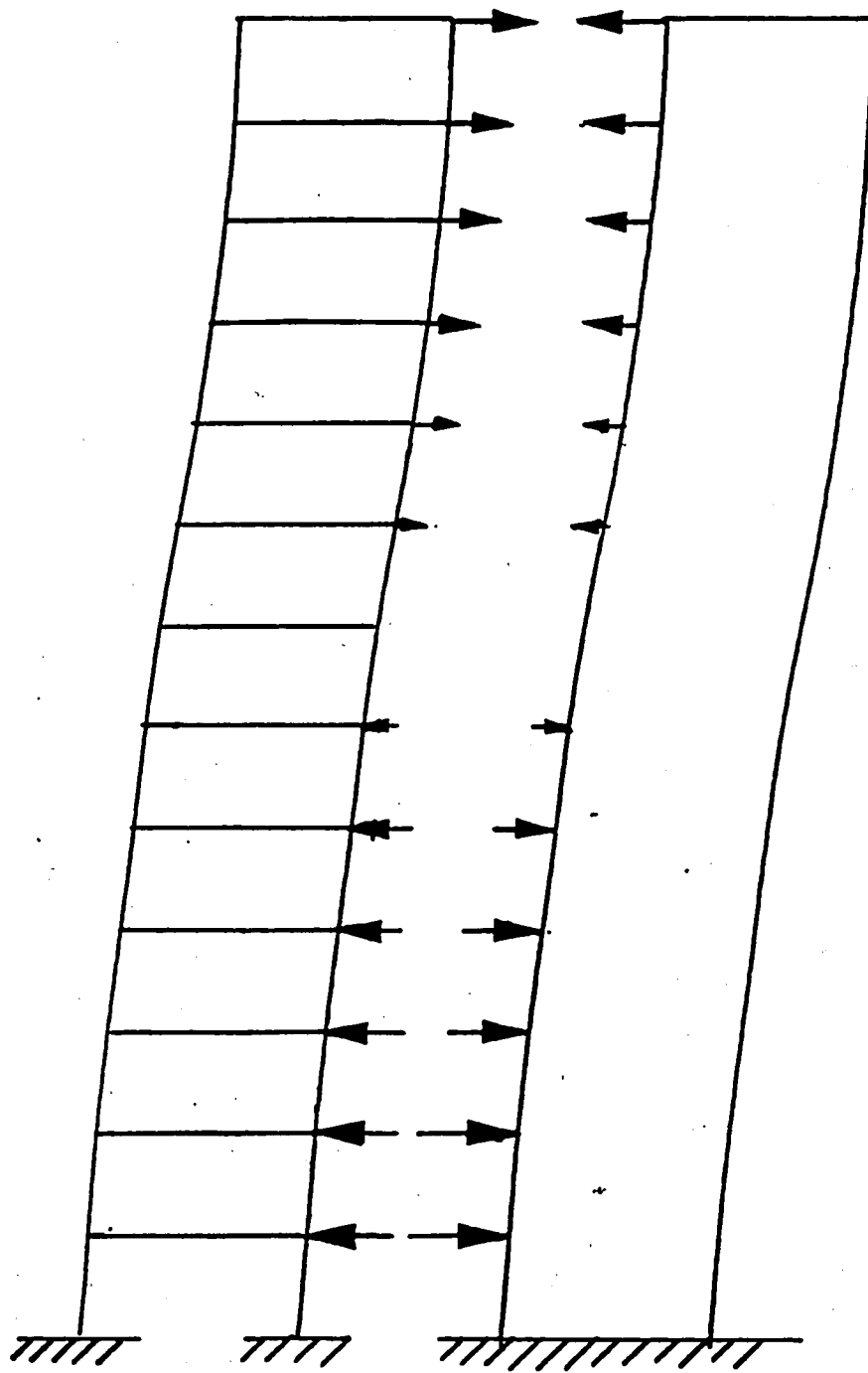


SHEAR WALL DEFORMATION

FIGURE 1



RIGID FRAME DEFORMATION
FIGURE 2



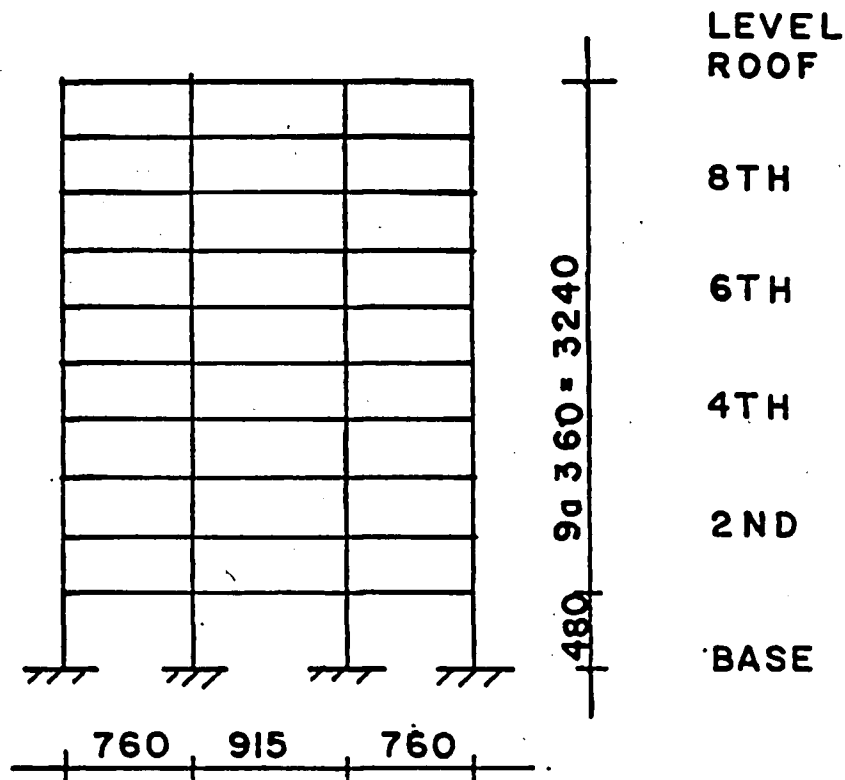
FRAME-SHEAR WALL

FIGURE 3

DESIGN LOADS
WIND LOAD
GRAVITY LOAD
ROOF
TYPICAL FLOOR

1197.5 Pa.
DEAD LOAD
7424.5 Pa.
6945.5 Pa.

LIVE L.
958 Pa.
2395 Pa.

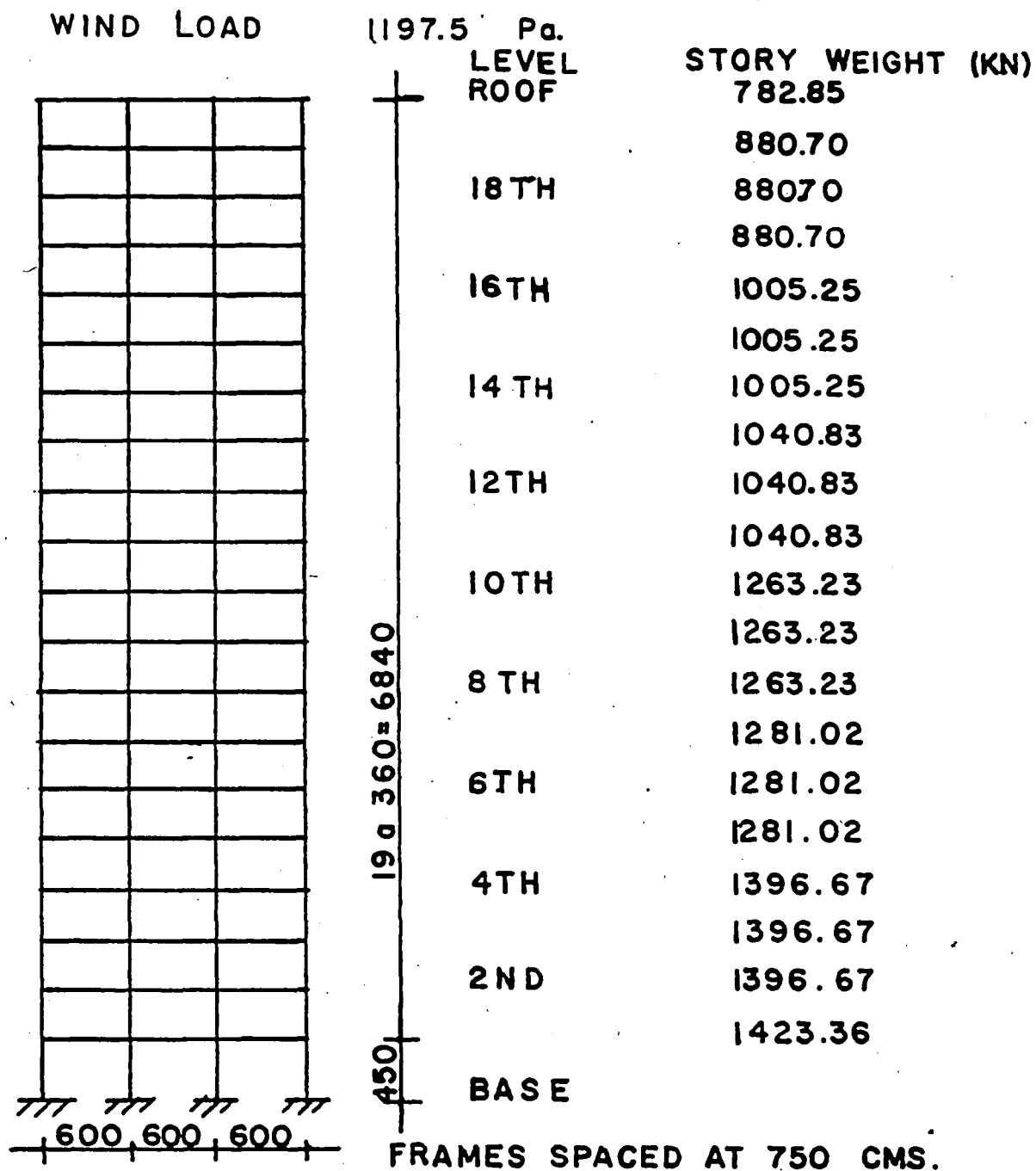


FRAMES SPACED AT 820 CENTIMETERS

FRAME 1
DIMENSIONS AND DESIGN LOADS
FIGURE 4

37.5 x 75		
55 x 55	37.5 x 75	65 x 65
55 x 55	37.5 x 75	65 x 65
60 x 60	40 x 80	70 x 70
60 x 60	40 x 80	70 x 70
65 x 65	42.5 x 85	75 x 75
65 x 65	42.5 x 85	75 x 75
70 x 70	42.5 x 85	80 x 80
70 x 70	42.5 x 85	80 x 80
75 x 75	42.5 x 85	85 x 85
75 x 75		85 x 85

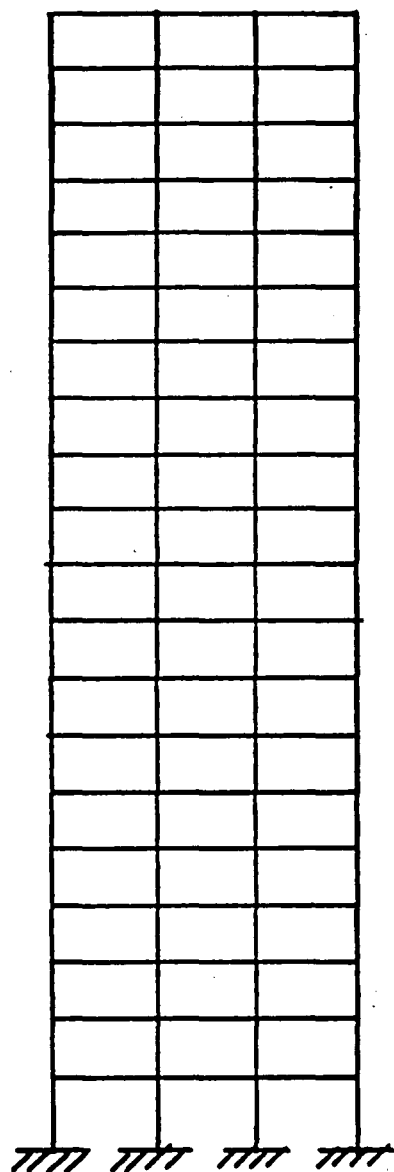
FRAME I
MEMBER SIZES
FIGURE 5



FRAME 2

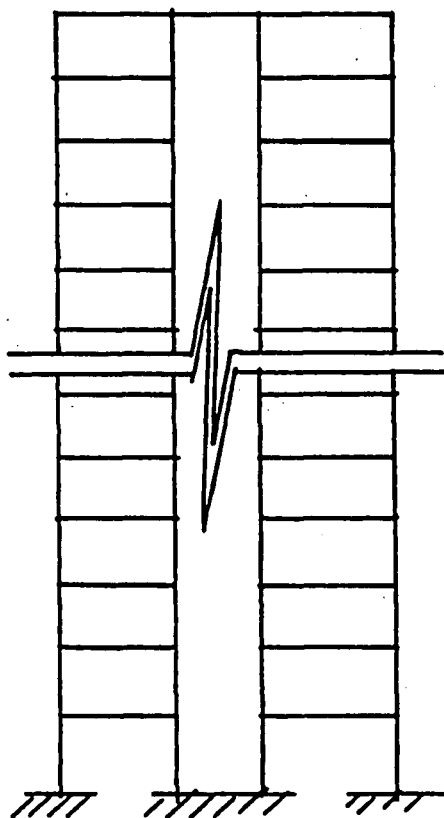
DIMENSIONS AND DESIGN LOADS

FIGURE 6

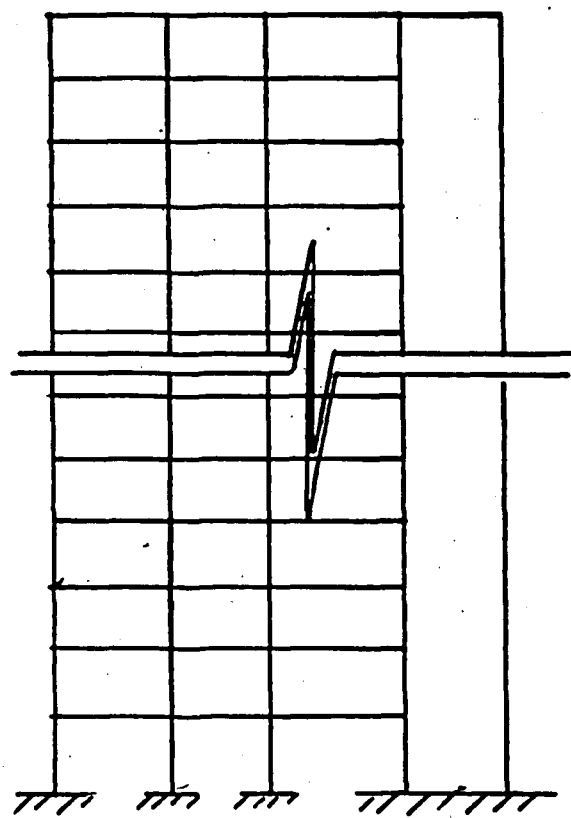


MEMBER SIZES (CENTIMETERS)		
COLUMNS		BEAMS
EXTERIOR	INTERIOR	
35 x 35	45 x 45	52.5 x 52.5
40 x 40	47.5 x 47.5	57.5 x 57.5
47.5 x 47.5	55 x 55	
52.5 x 52.5	60 x 60	62.5 x 62.5
55 x 55	65 x 65	
62.5 x 62.5	75 x 75	65 x 65
65 x 65	77.5 x 77.5	
65 x 65	77.5 x 77.5	

FRAME 2
MEMBER SIZES
FIGURE 7

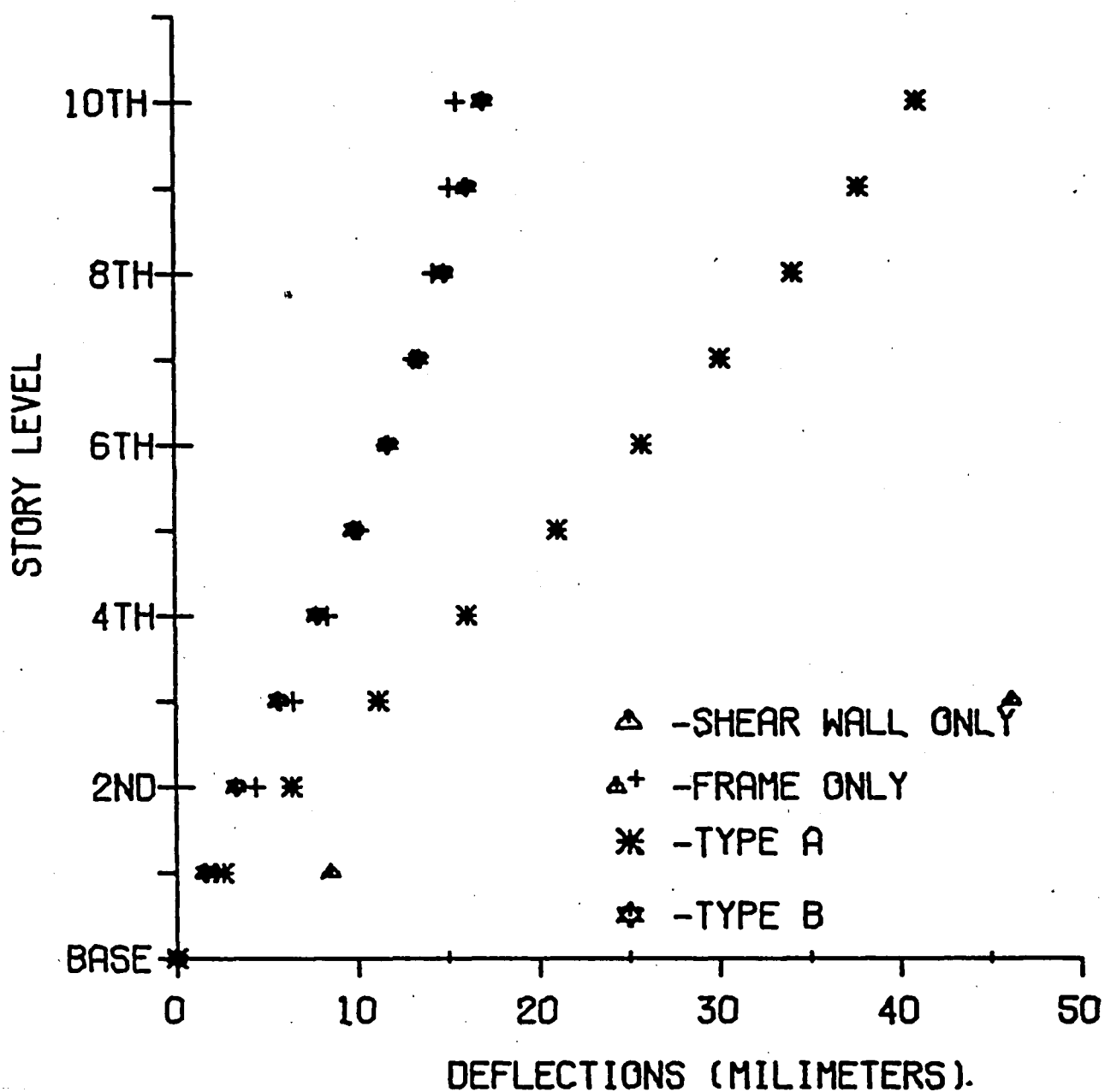


TYPE A



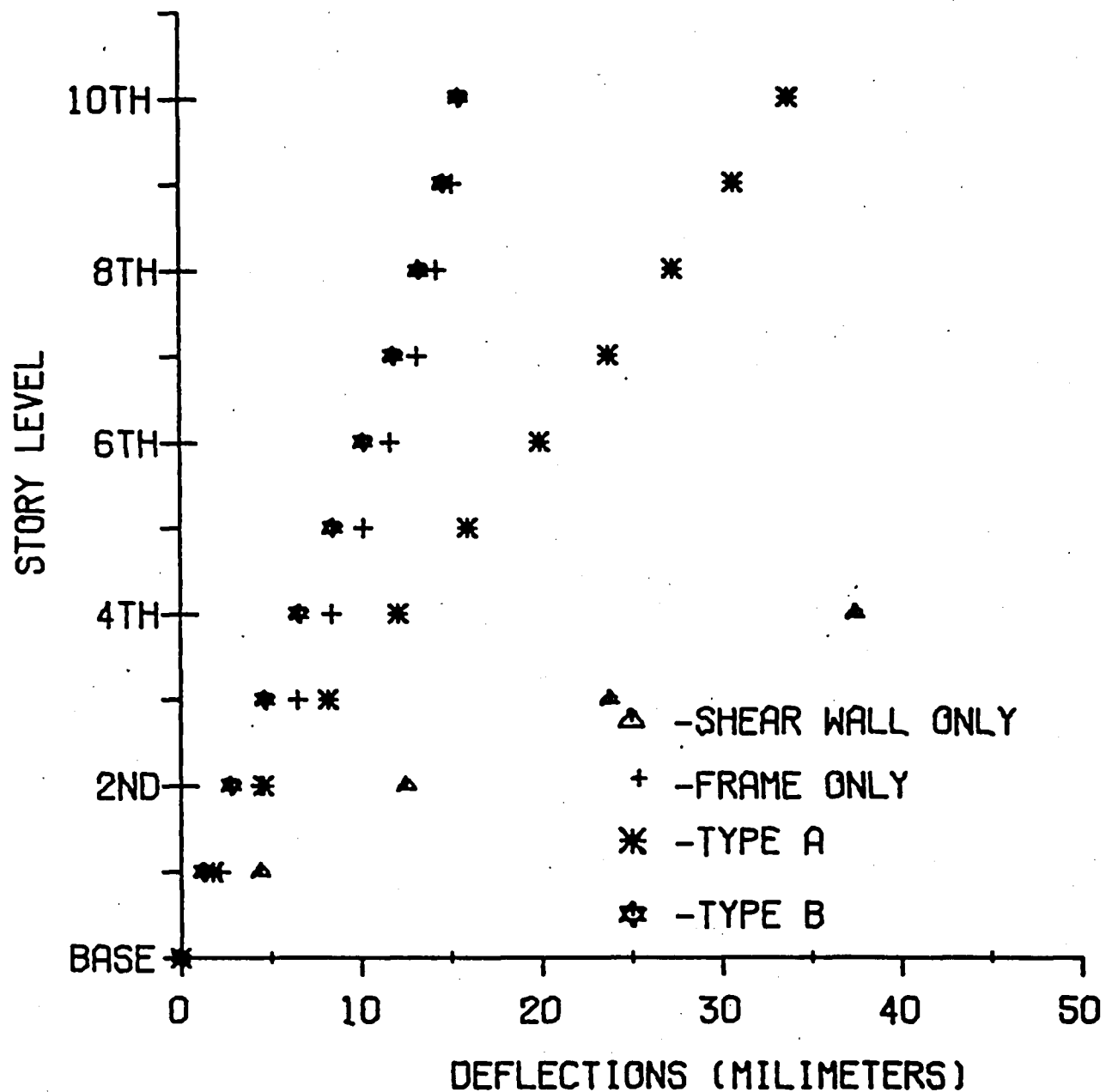
TYPE B

FRAME-SHEAR WALL CONFIGURATIONS FIGURE 8.



DEFLECTION PROFILES
 FRAME 1-SHEAR WALL A
 S. W. LENGTH=244 CM

FIGURE 9

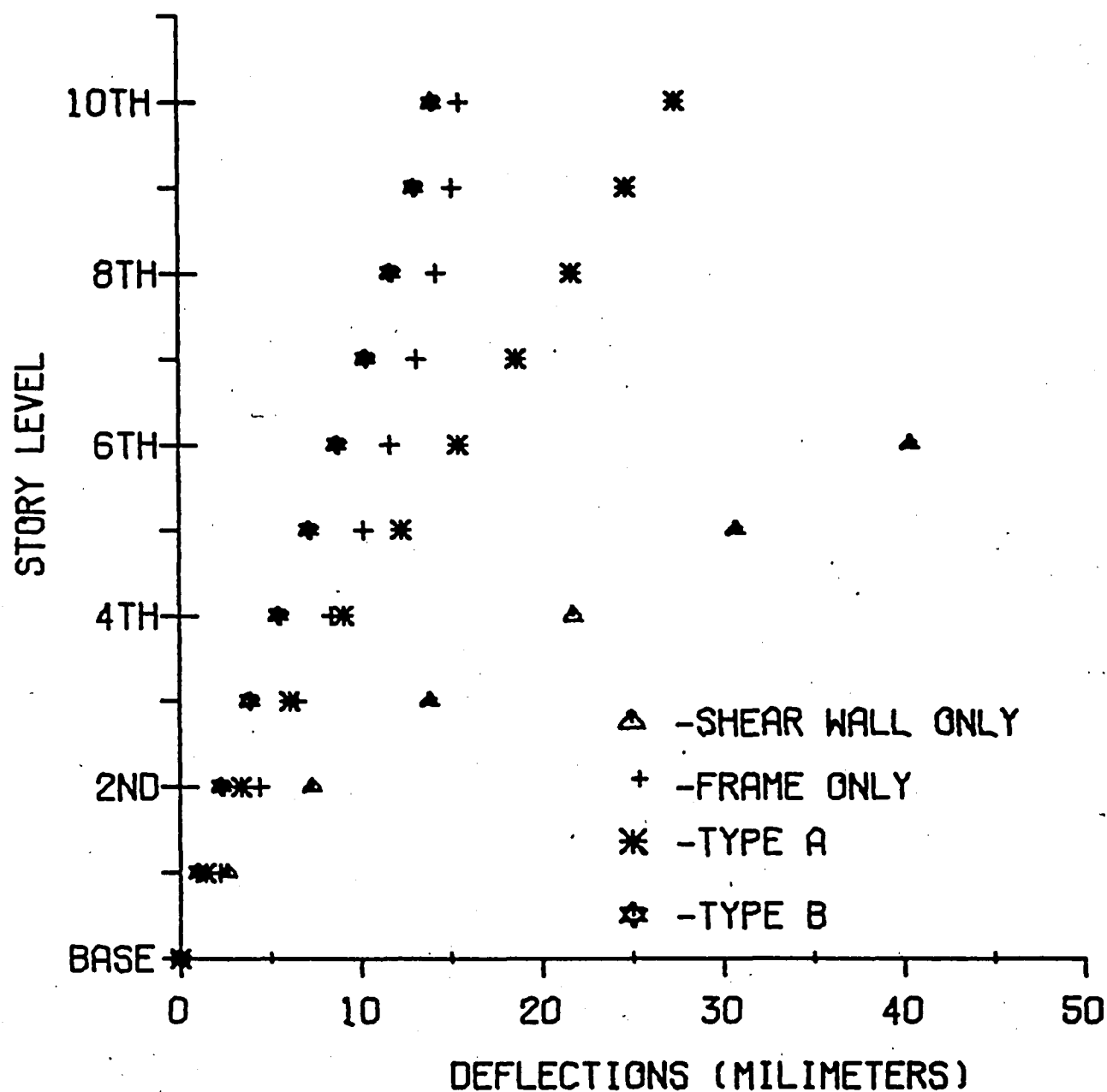


DEFLECTION PROFILES

FRAME 1-SHEAR WALL B

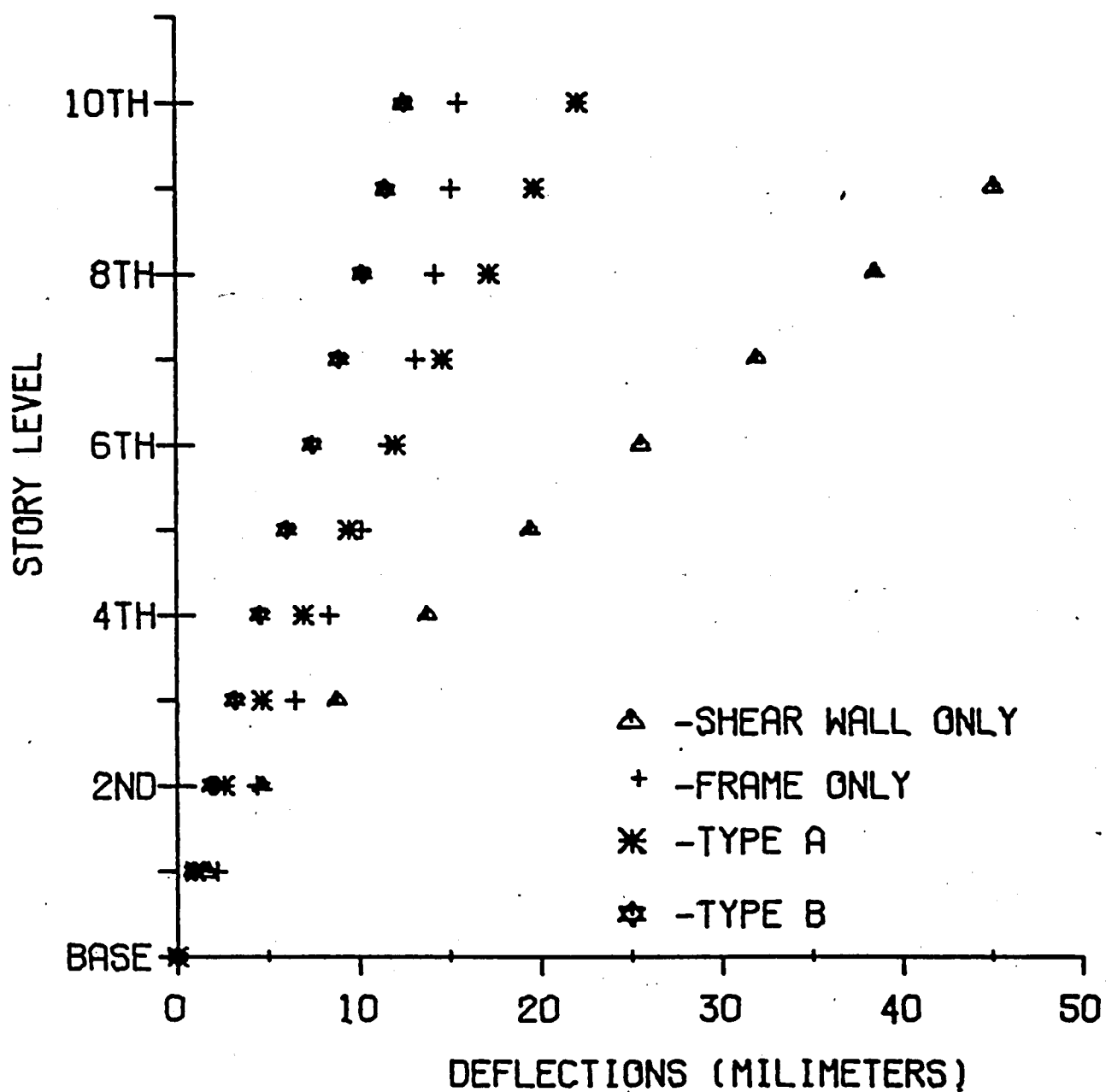
S. W. LENGTH=305 cm

FIGURE 10



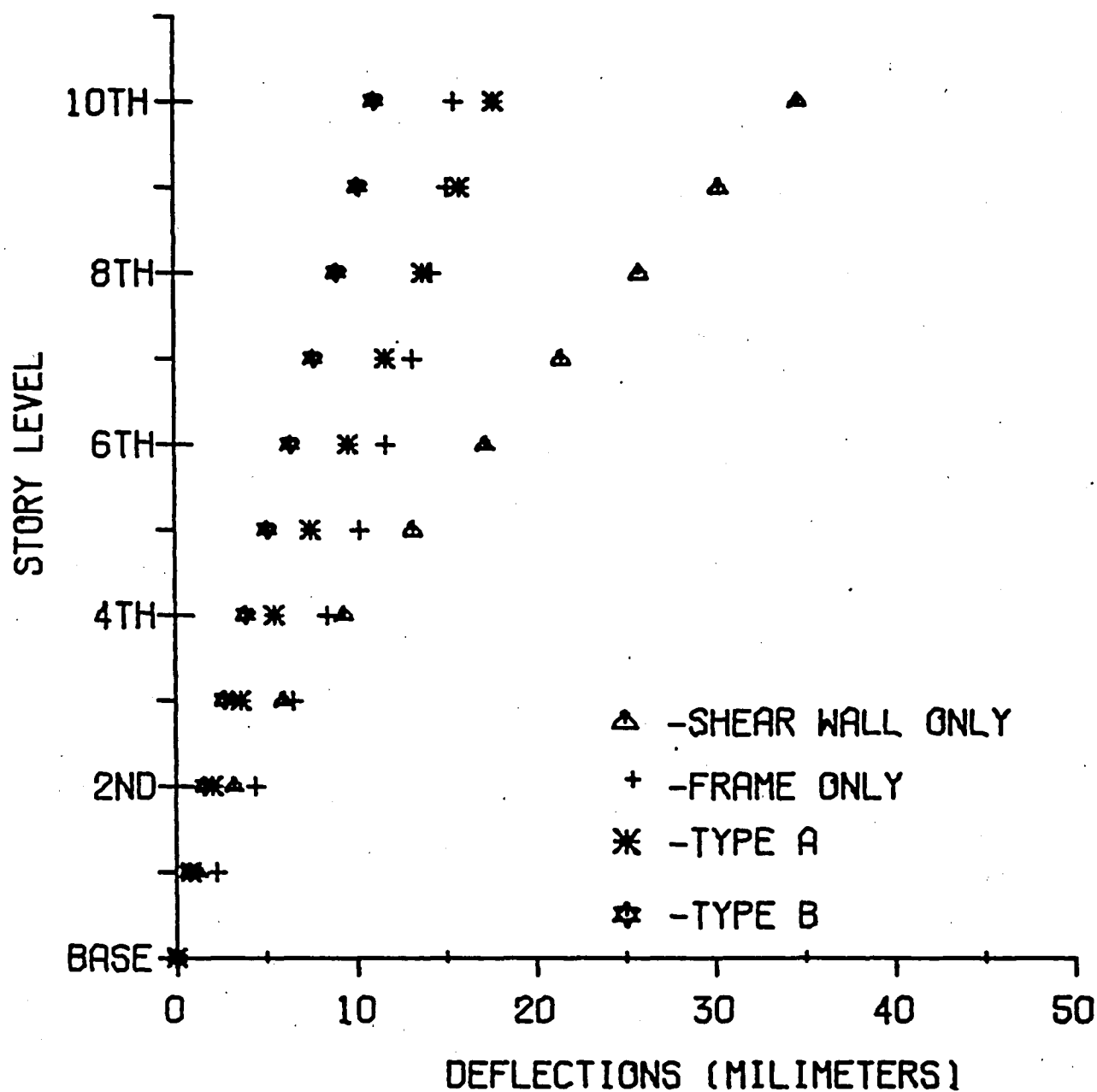
DEFLECTION PROFILES
 FRAME 1-SHEAR WALL C
 S. W. LENGTH=366 CM

FIGURE 11



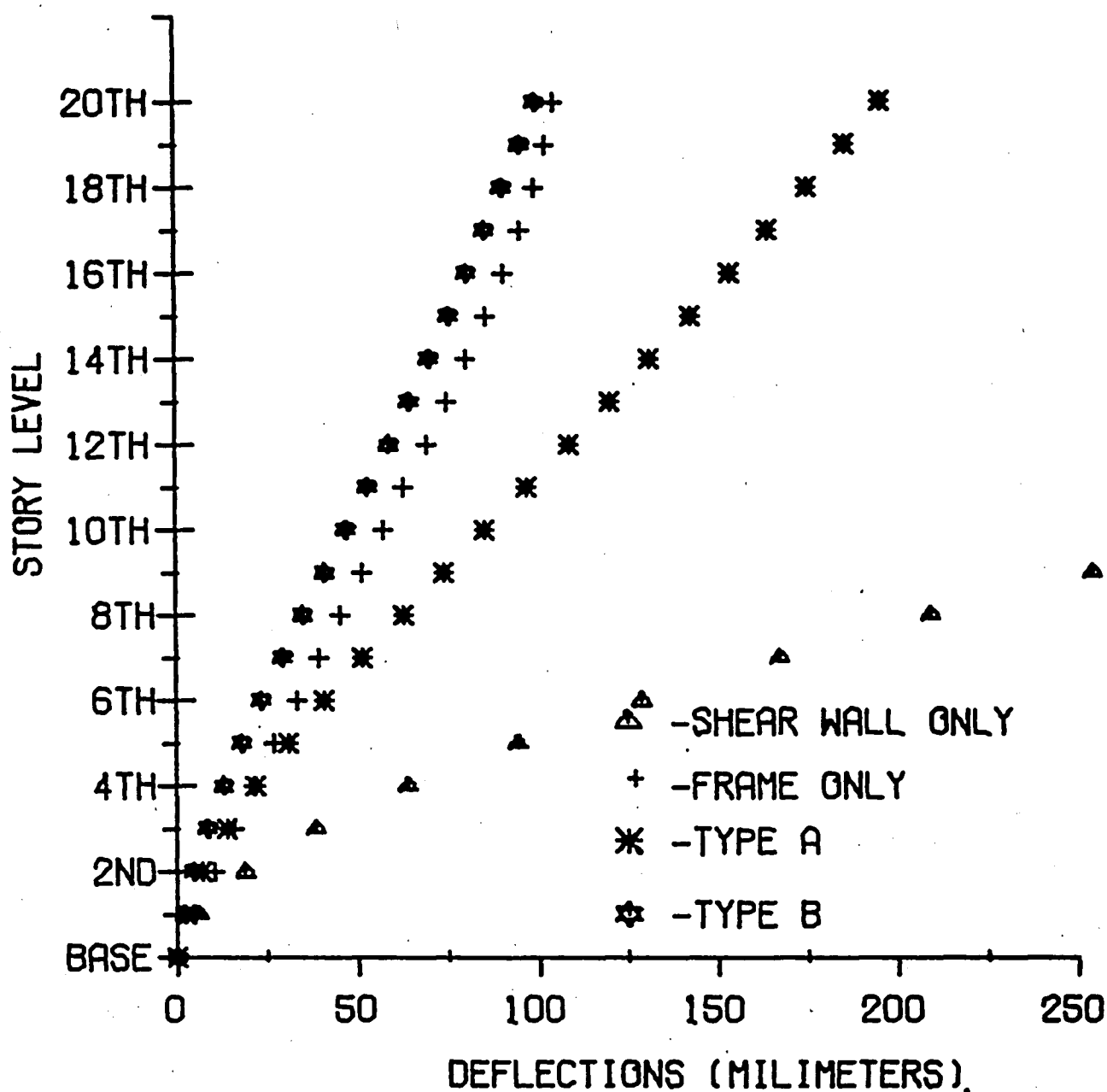
DEFLECTION PROFILES
 FRAME 1-SHEAR WALL D
 S. W. LENGTH=427 cm

FIGURE 12



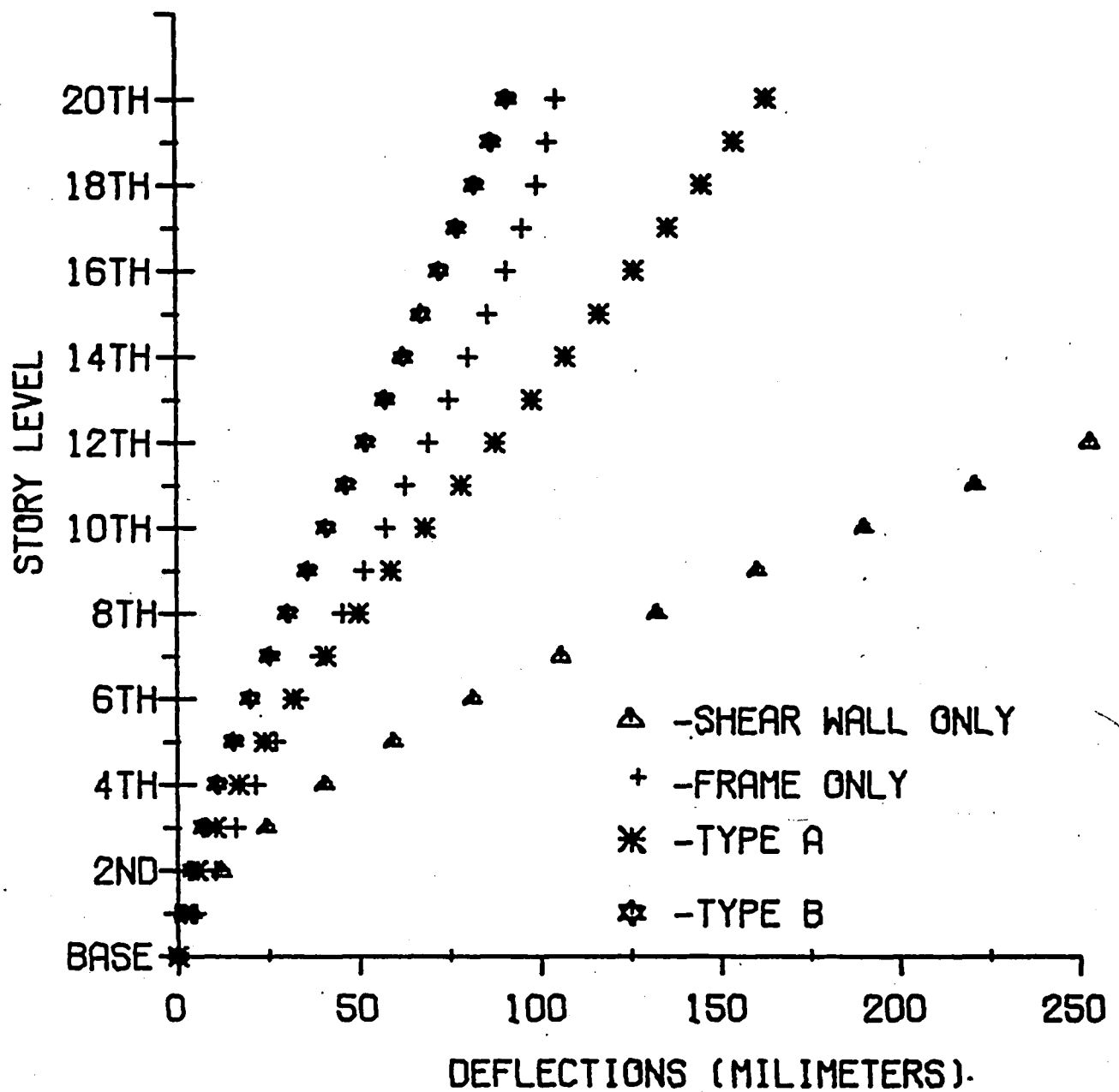
DEFLECTION PROFILES
FRAME 1-SHEAR WALL E
S. W. LENGTH=488 cm

FIGURE 13



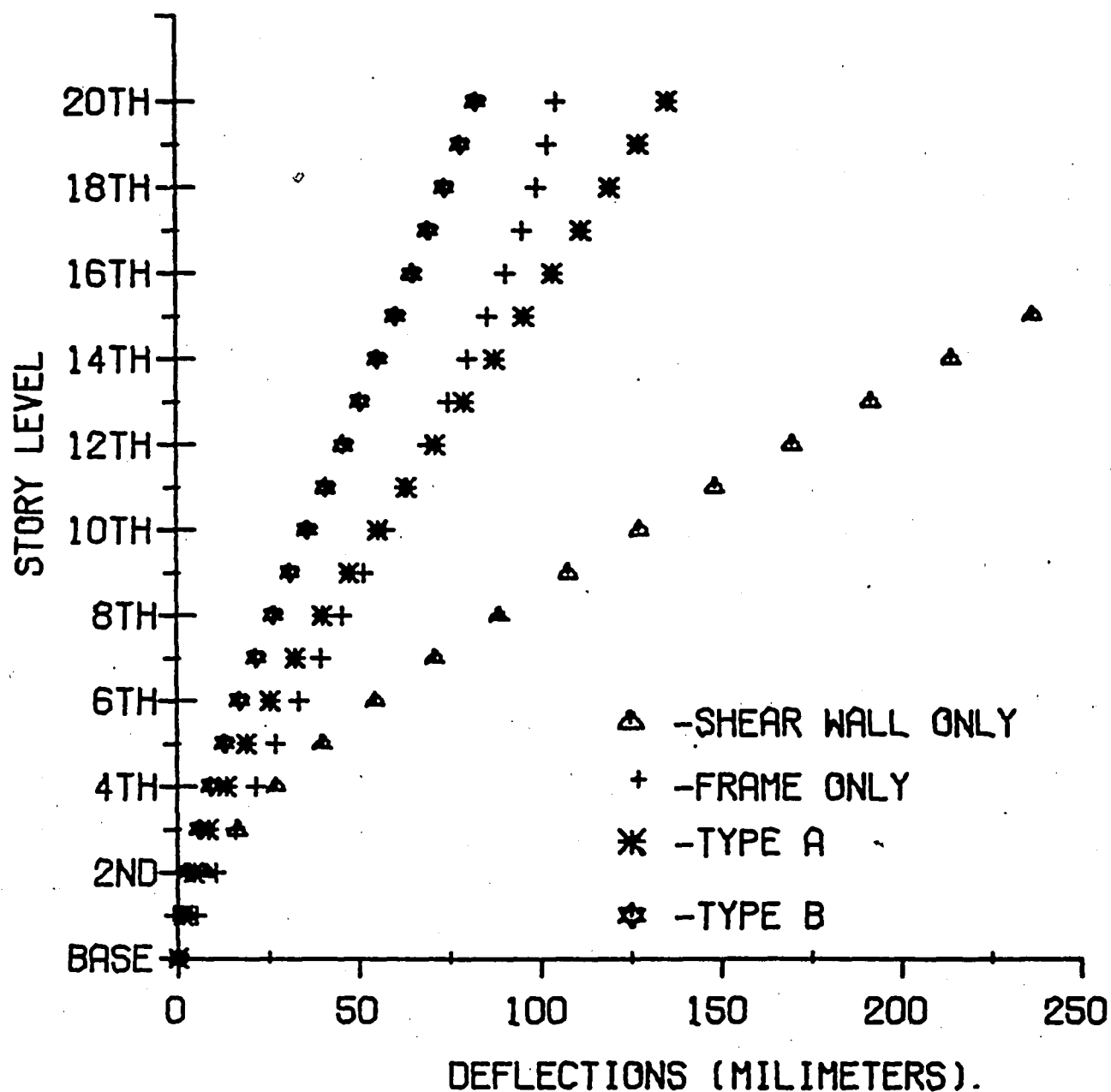
DEFLECTION PROFILES
 FRAME 2-SHEAR WALL A
 S. W. LENGTH=366 cm

FIGURE 14



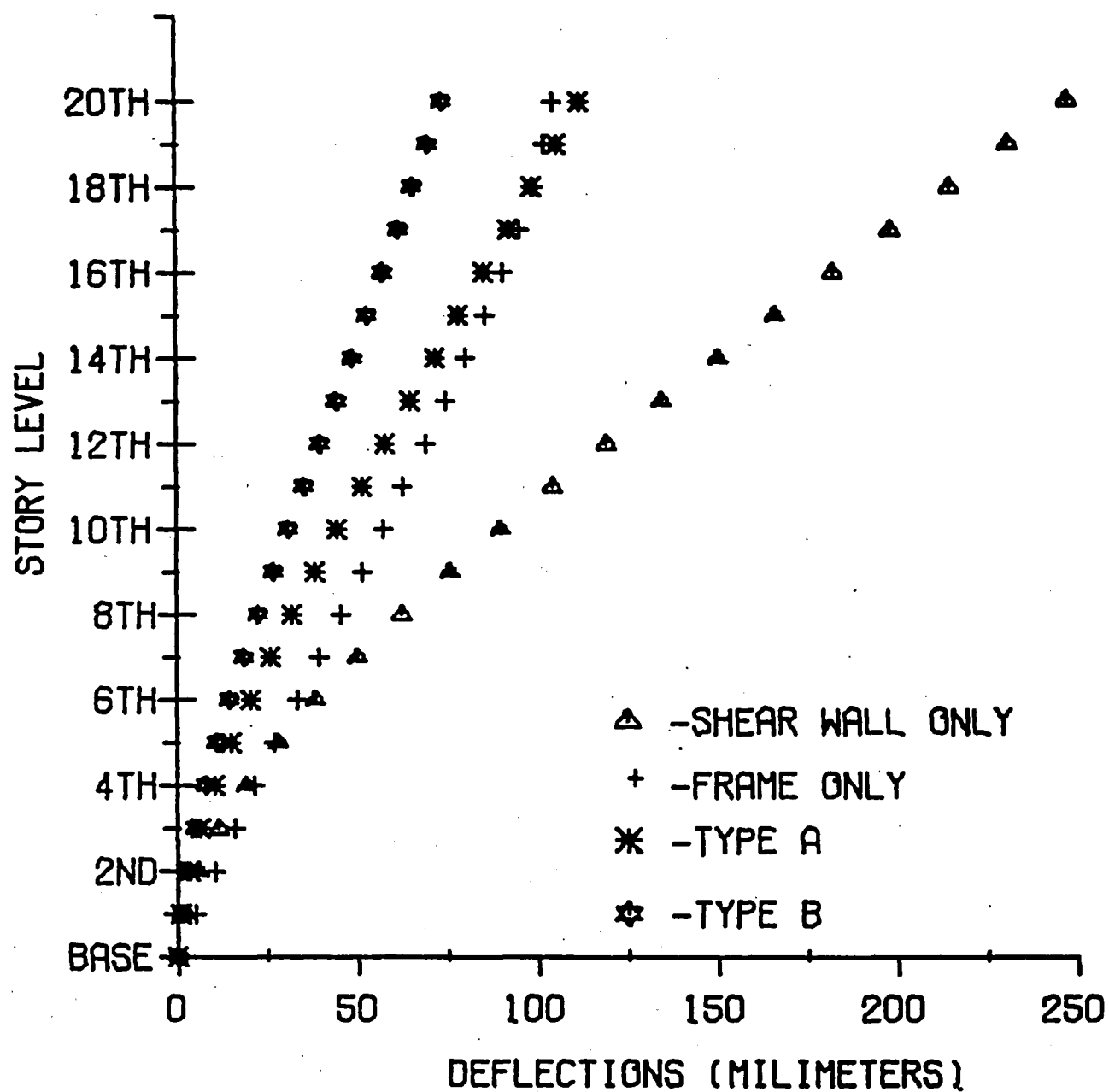
DEFLECTION PROFILES
 FRAME 2-SHEAR WALL B
 S. W. LENGTH=427 CM

FIGURE 15



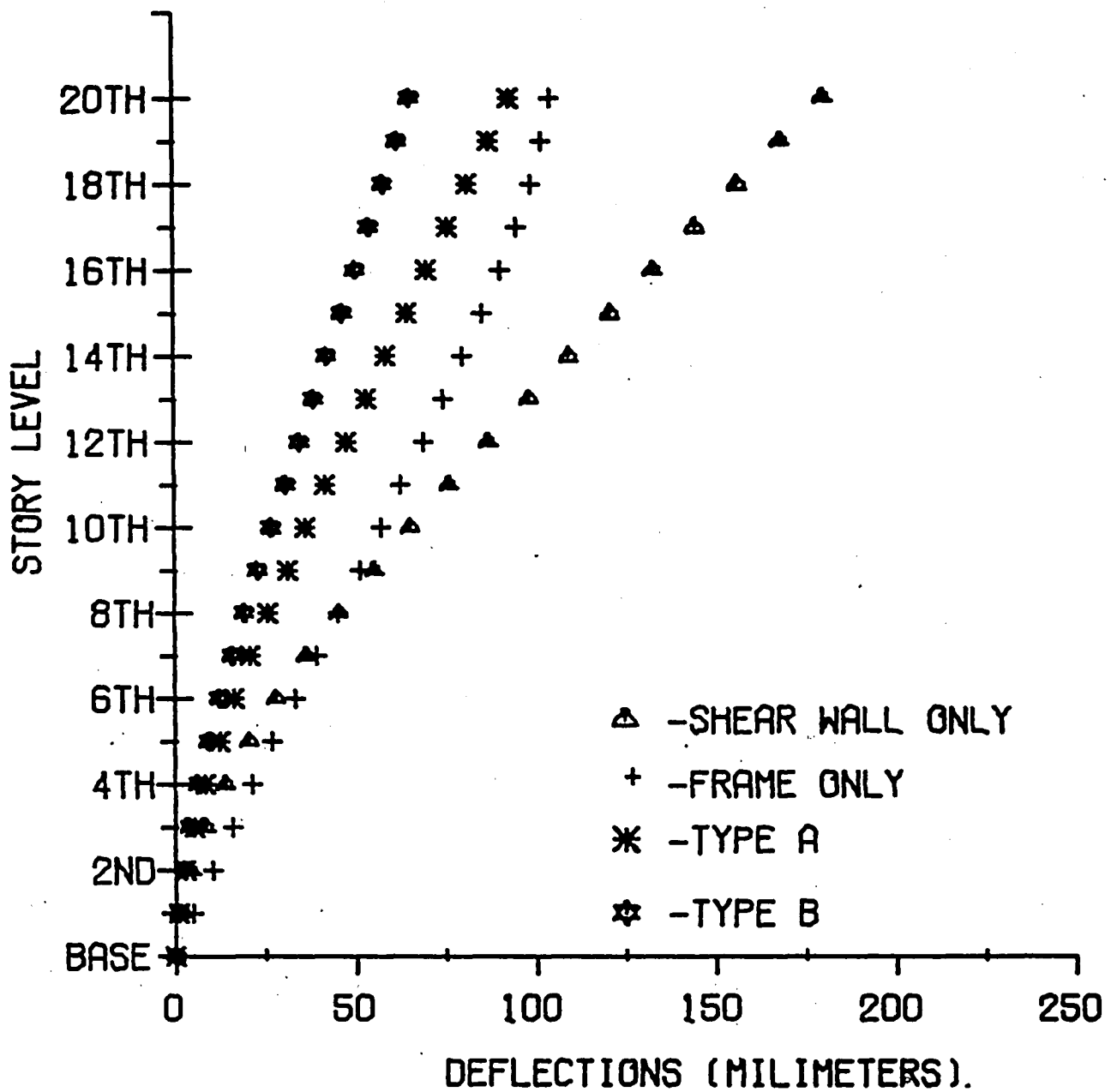
DEFLECTION PROFILES
 FRAME 2-SHEAR WALL C
 S. W. LENGTH=488 CM

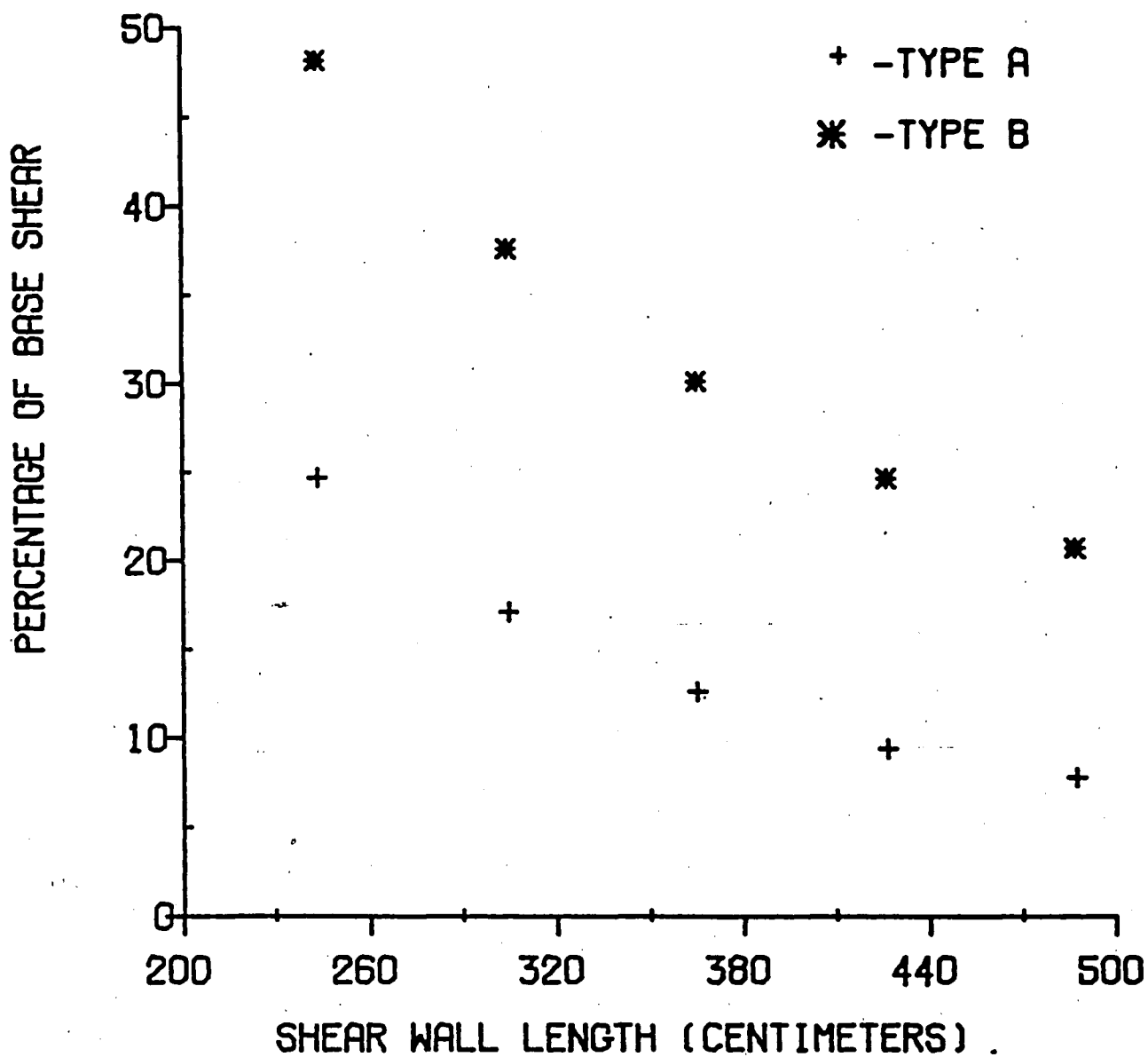
FIGURE 16



DEFLECTION PROFILES
FRAME 2-SHEAR WALL D
S. W. LENGTH=549 cm

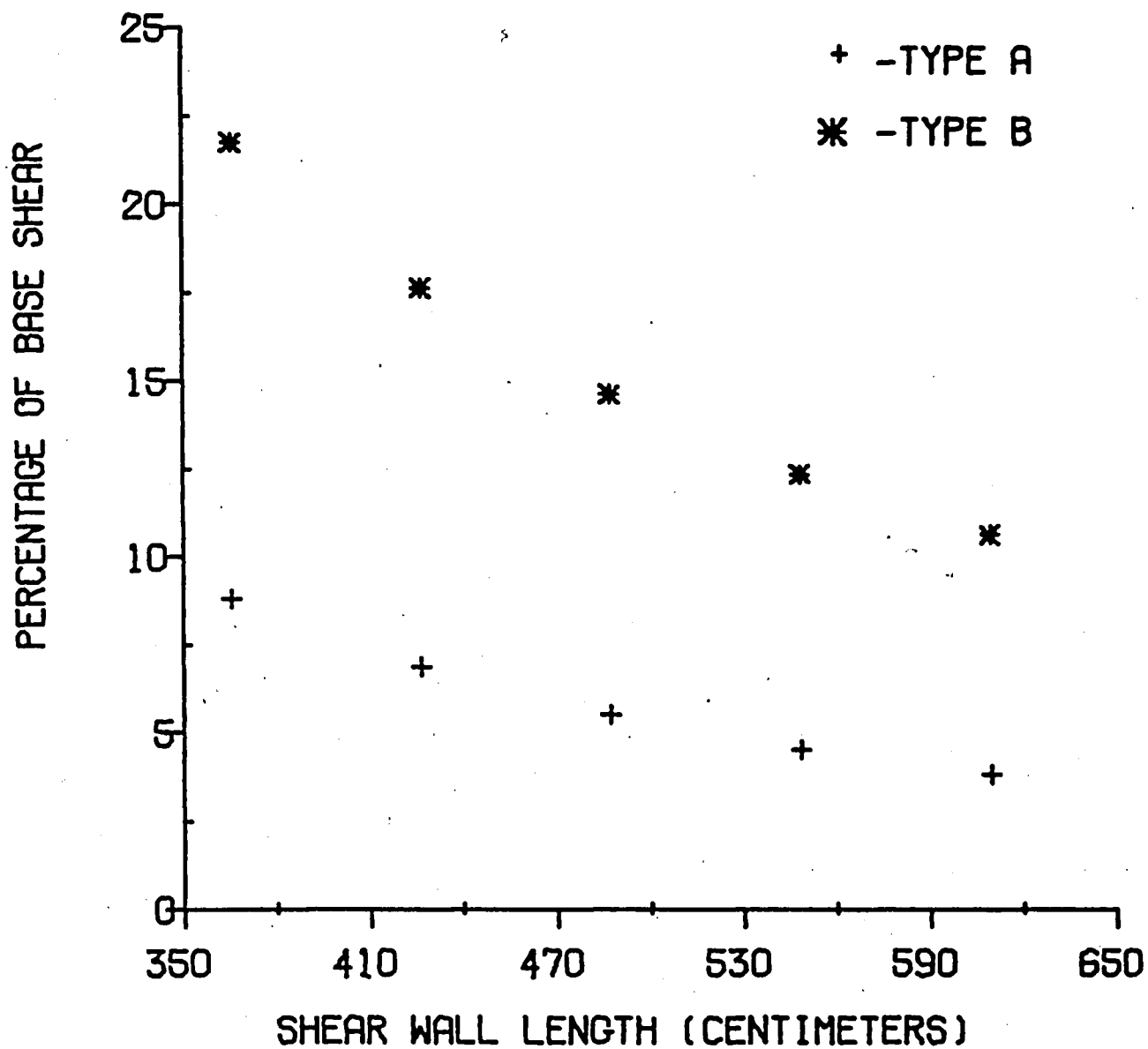
FIGURE 17





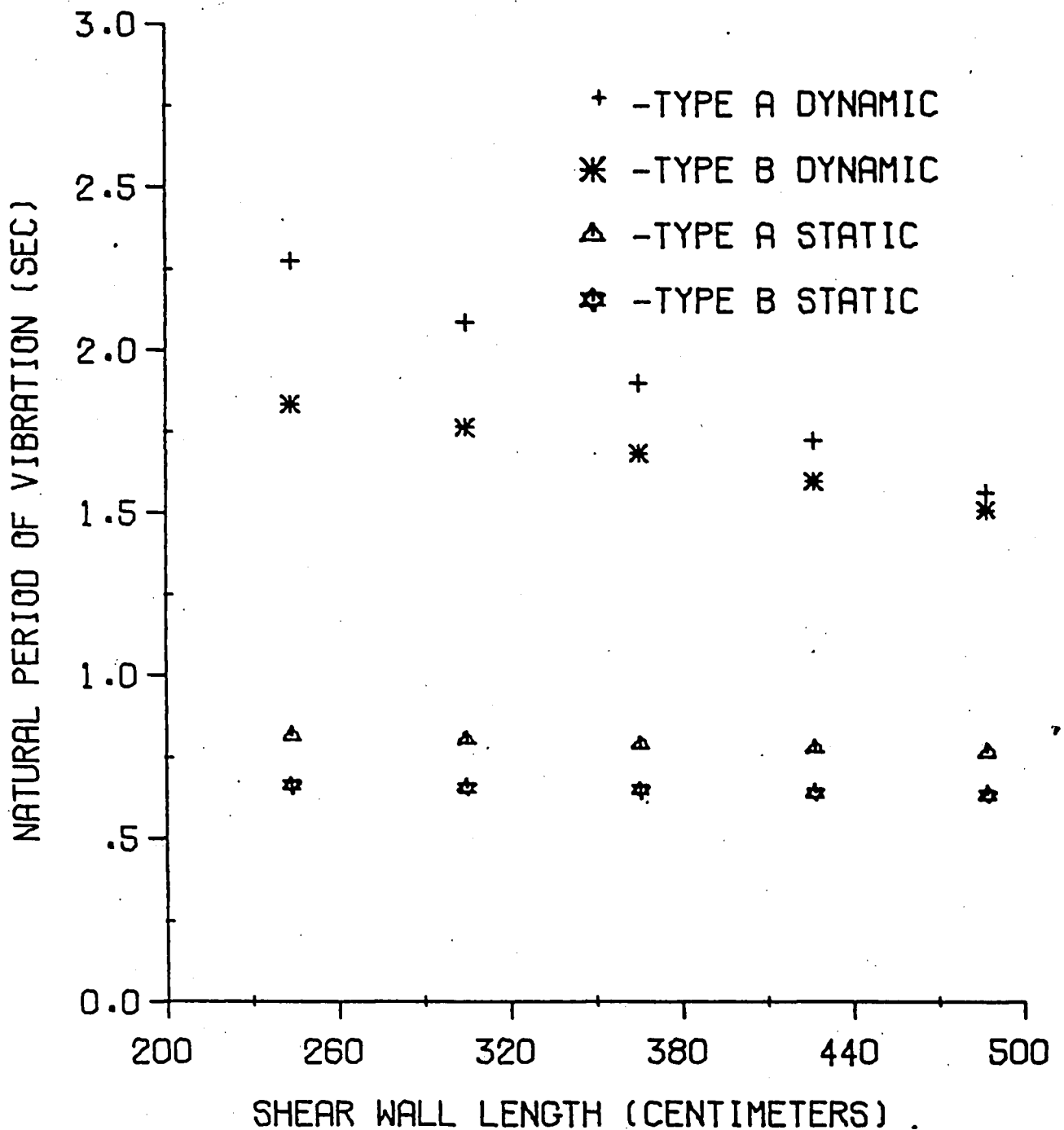
PERCENTAGES OF BASE SHEAR
ON FRAME 1 FOR CHOSEN
DIMENSIONS OF SHEAR WALL

FIGURE 19



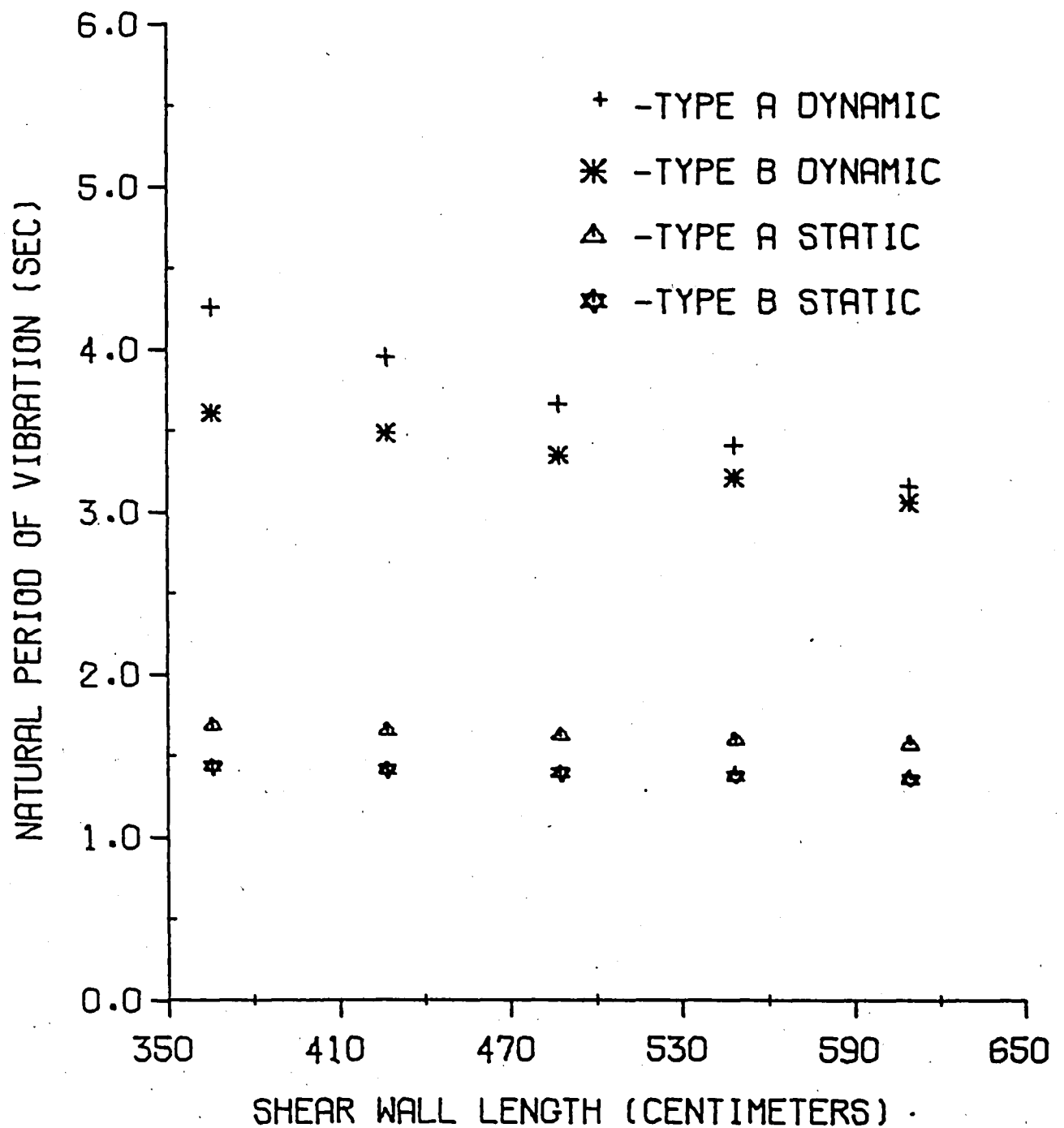
PERCENTAGES OF BASE SHEAR
ON FRAME 2 FOR CHOSEN
DIMENSIONS OF SHEAR WALL

FIGURE 20



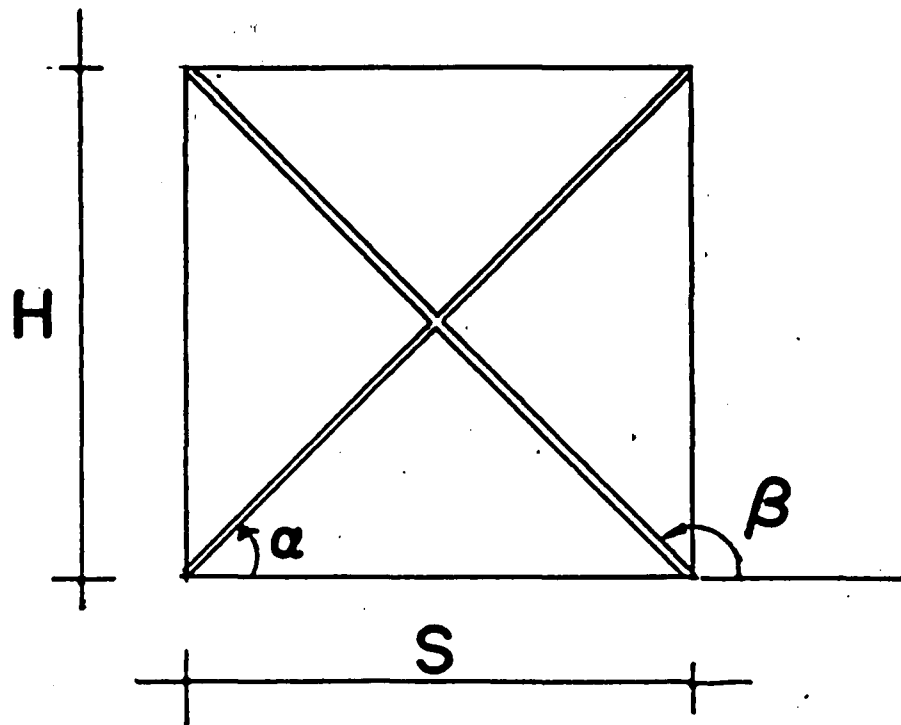
NATURAL PERIODS OF VIBRATION
FOR CHOSEN COMBINATIONS OF
FRAME 1 AND SHEAR WALLS

FIGURE 21



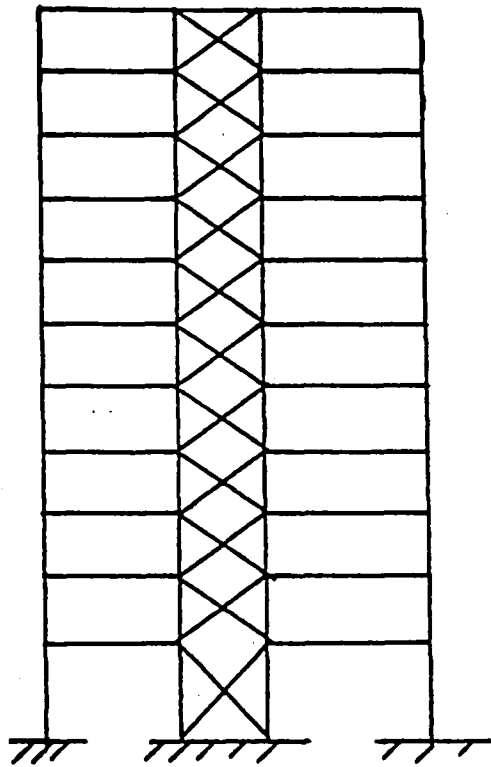
NATURAL PERIODS OF VIBRATION
FOR CHOSEN COMBINATIONS OF
FRAME 2 AND SHEAR WALLS

FIGURE 22

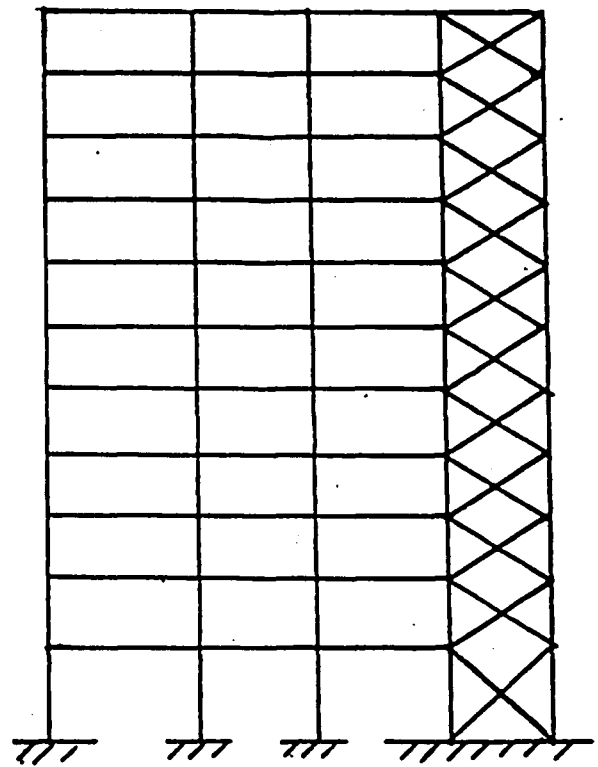


FRAME-SHEAR WALL	H (CENTIMETERS)	S-VARIABLE (CENTIMETERS)
FRAME 1	480	240-480
FRAME 1	360	240-480
FRAME 2	450	300-600
FRAME 2	360	300-600

**SHEAR WALL PANEL
ASSUMED CRACK PATTERN
FIGURE 23**



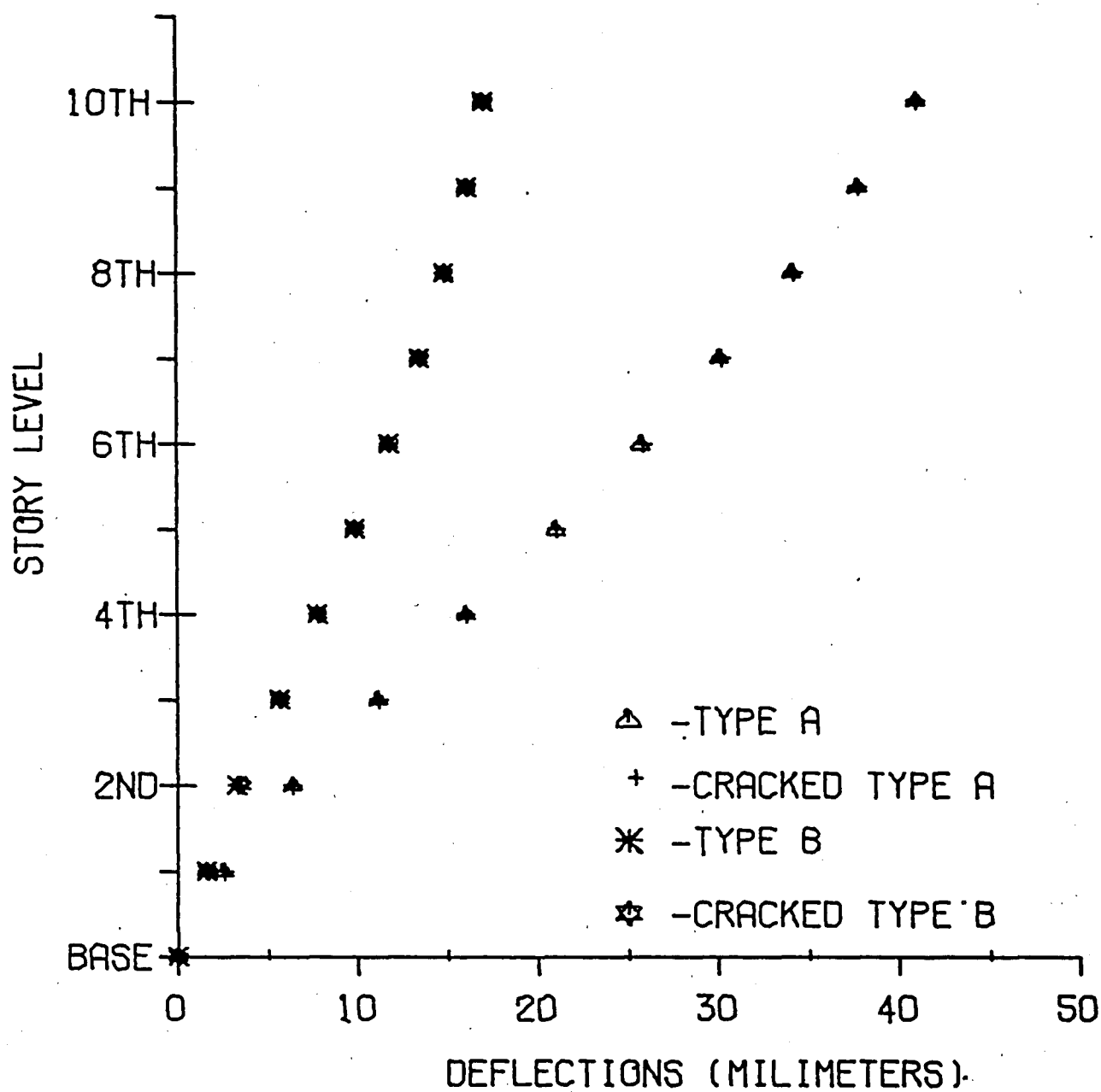
CRACKED TYPE A



CRACKED TYPE B

FRAME-CRACKED SHEAR WALL CONFIGURATIONS

FIGURE 24

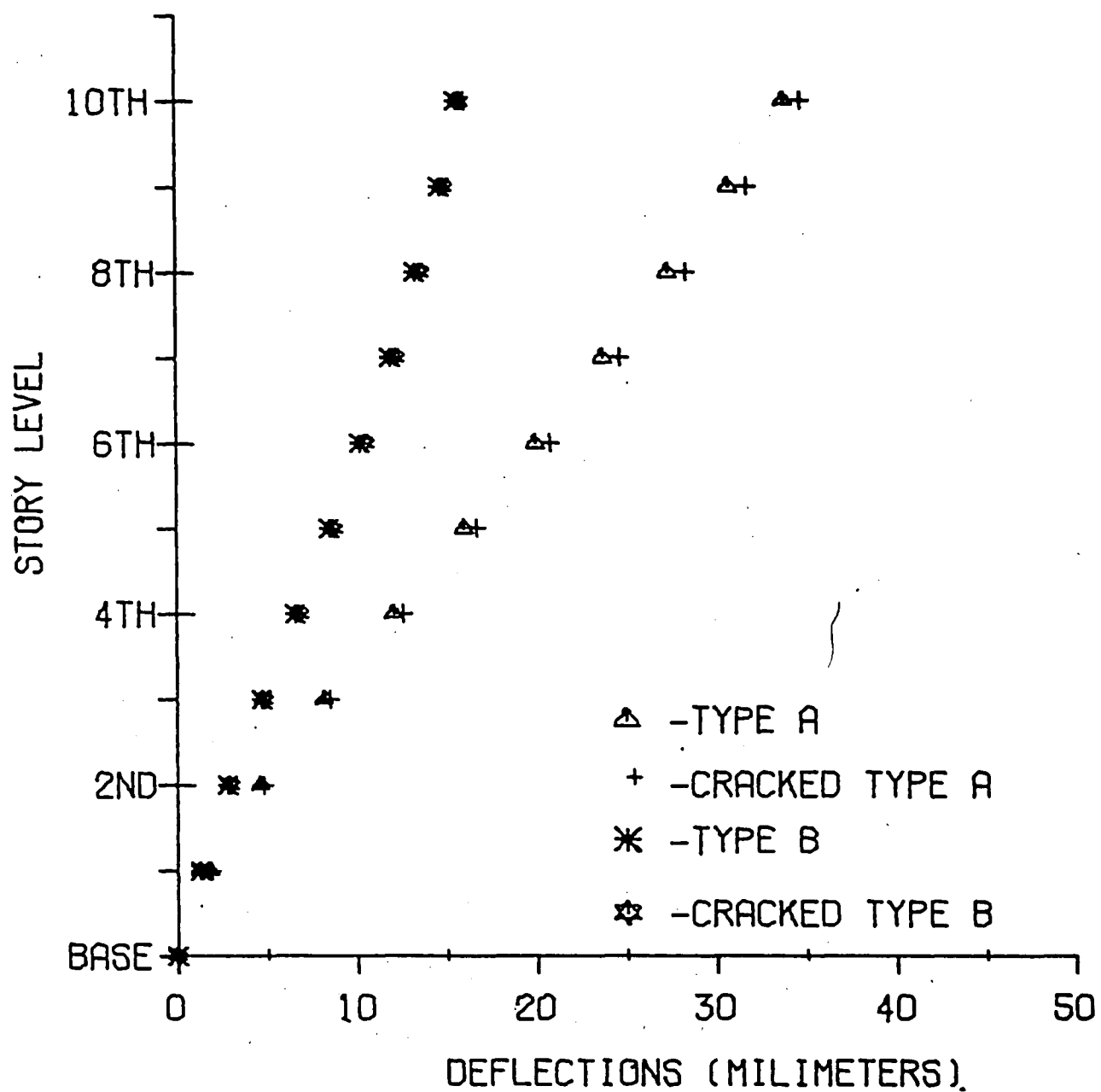


DEFLECTION PROFILES

FRAME 1 - CRACKED S. W. A

$$E_s = 0.25E_c$$

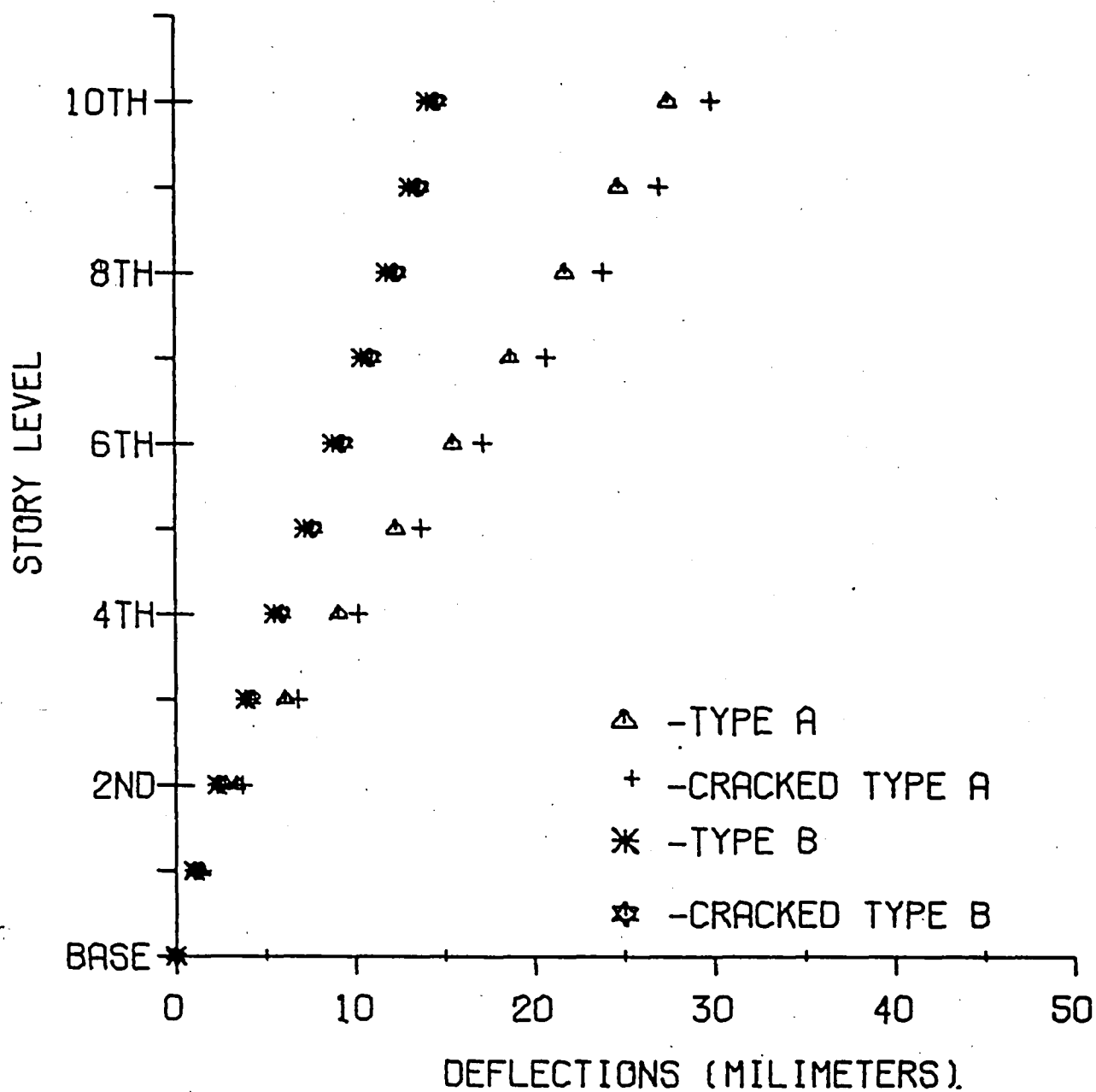
FIGURE 25



DEFLECTION PROFILES
 FRAME 1 - CRACKED S. W. B

$$E_s = 0.25E_c$$

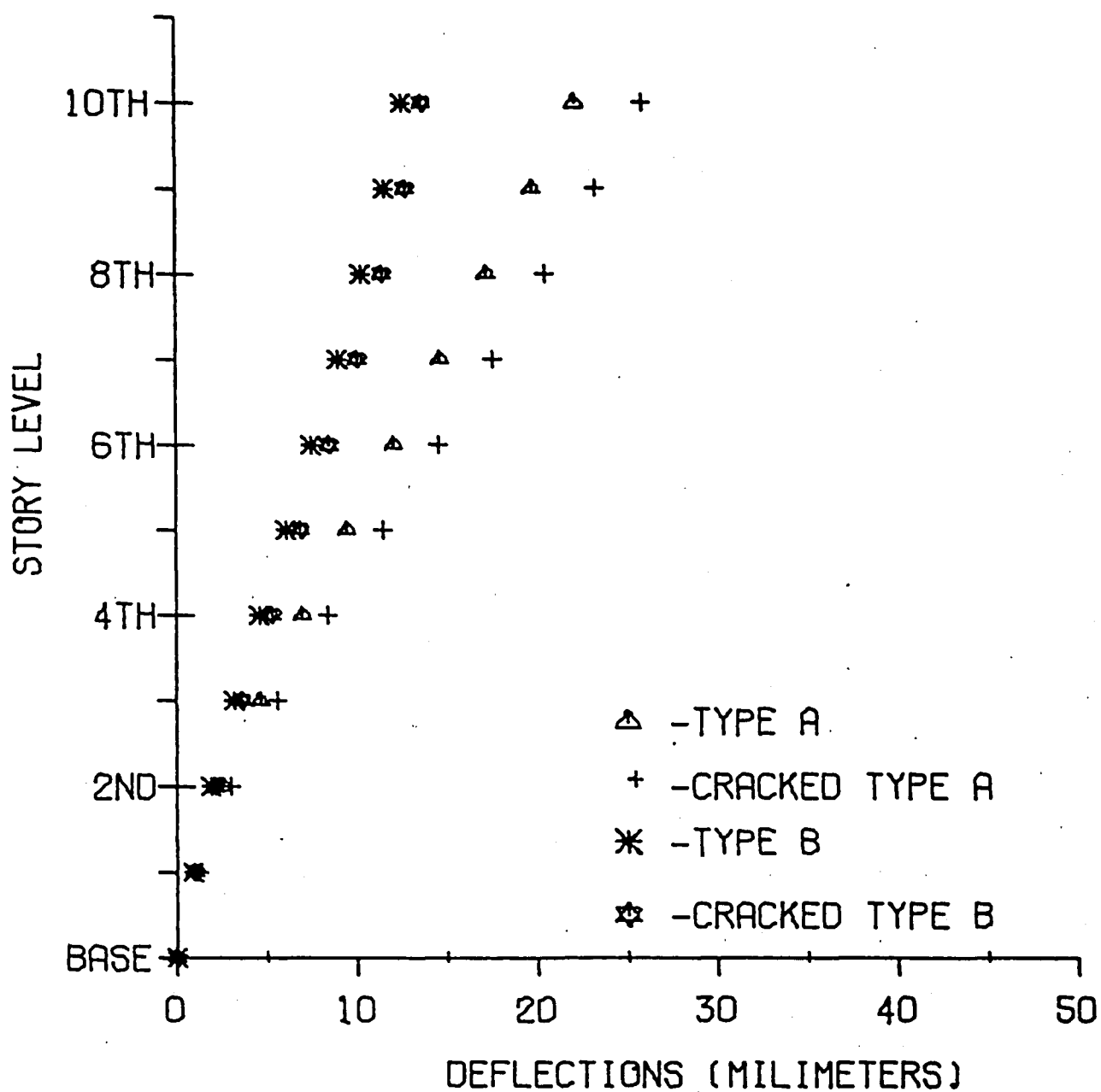
FIGURE 26



DEFLECTION PROFILES
 FRAME 1 - CRACKED S. W. C

$$E_s = 0.25E_c$$

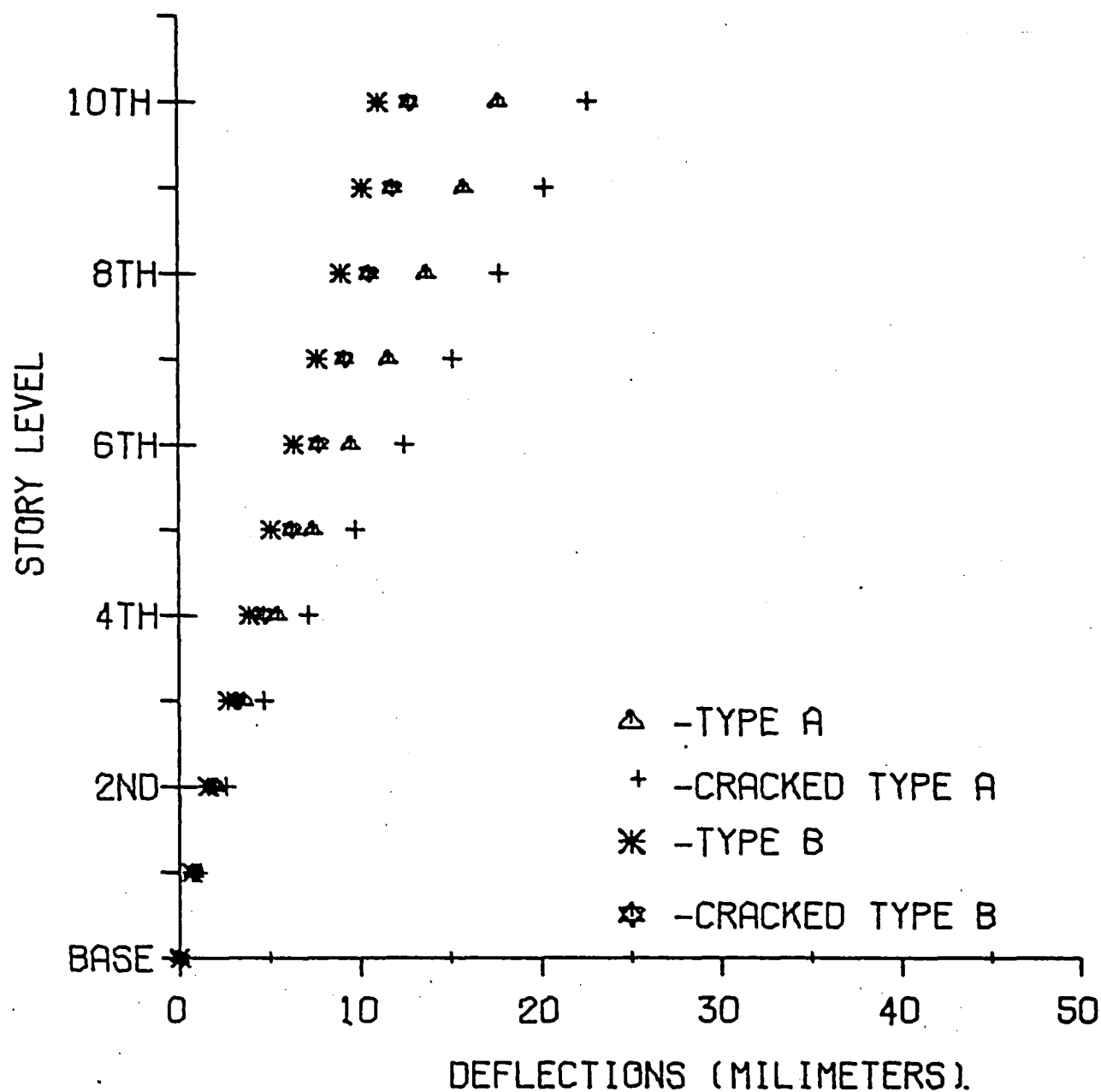
FIGURE 27



DEFLECTION PROFILES
 FRAME 1 - CRACKED S. W. D

$$E_s = 0.25E_c$$

FIGURE 28

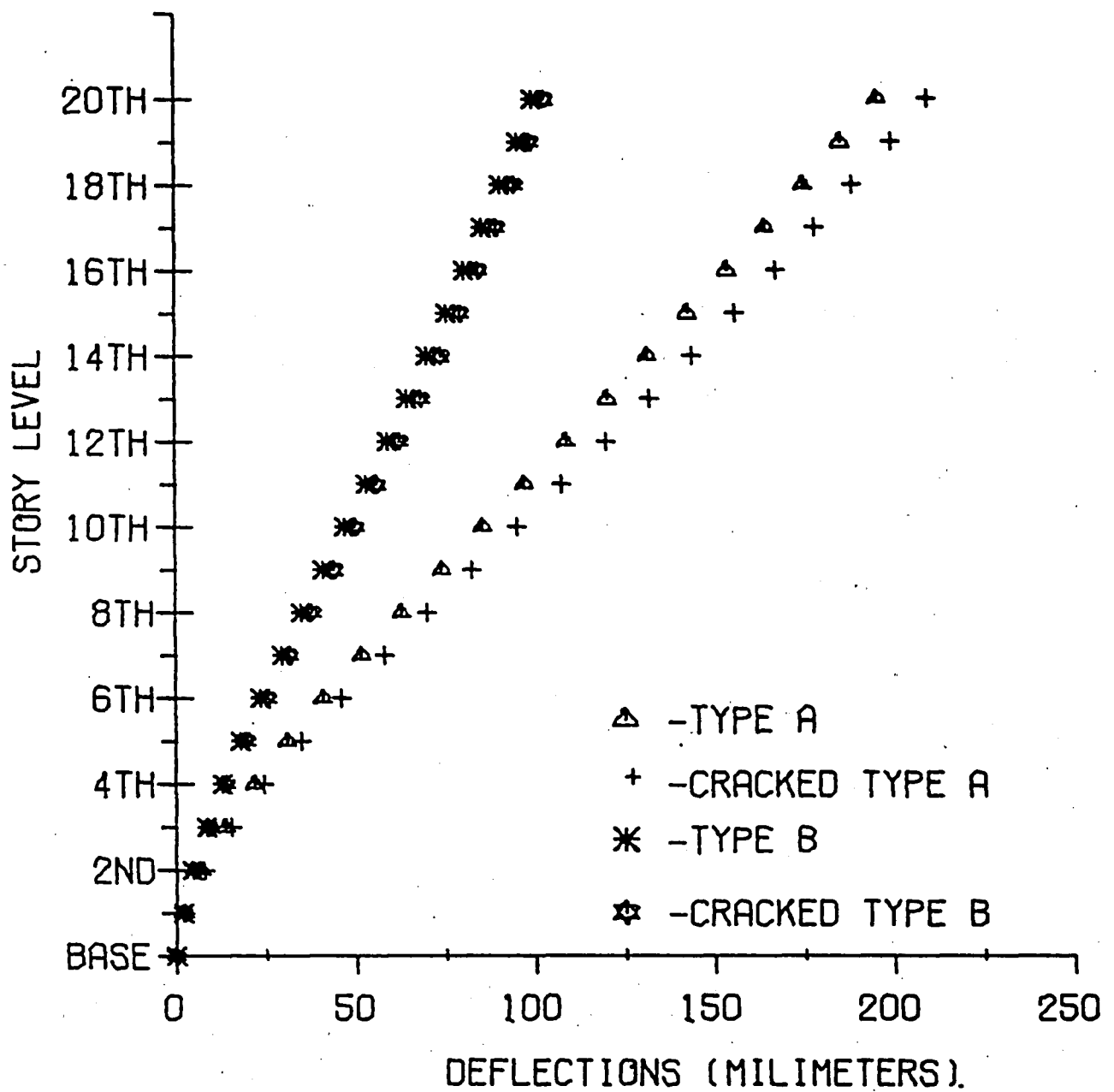


DEFLECTION PROFILES

FRAME 1 - CRACKED S. W. E

$$E_s = 0.25E_c$$

FIGURE 29



DEFLECTION PROFILES

FRAME 2 - CRACKED S. W. A

$$E_s = 0.25E_c$$

FIGURE 30

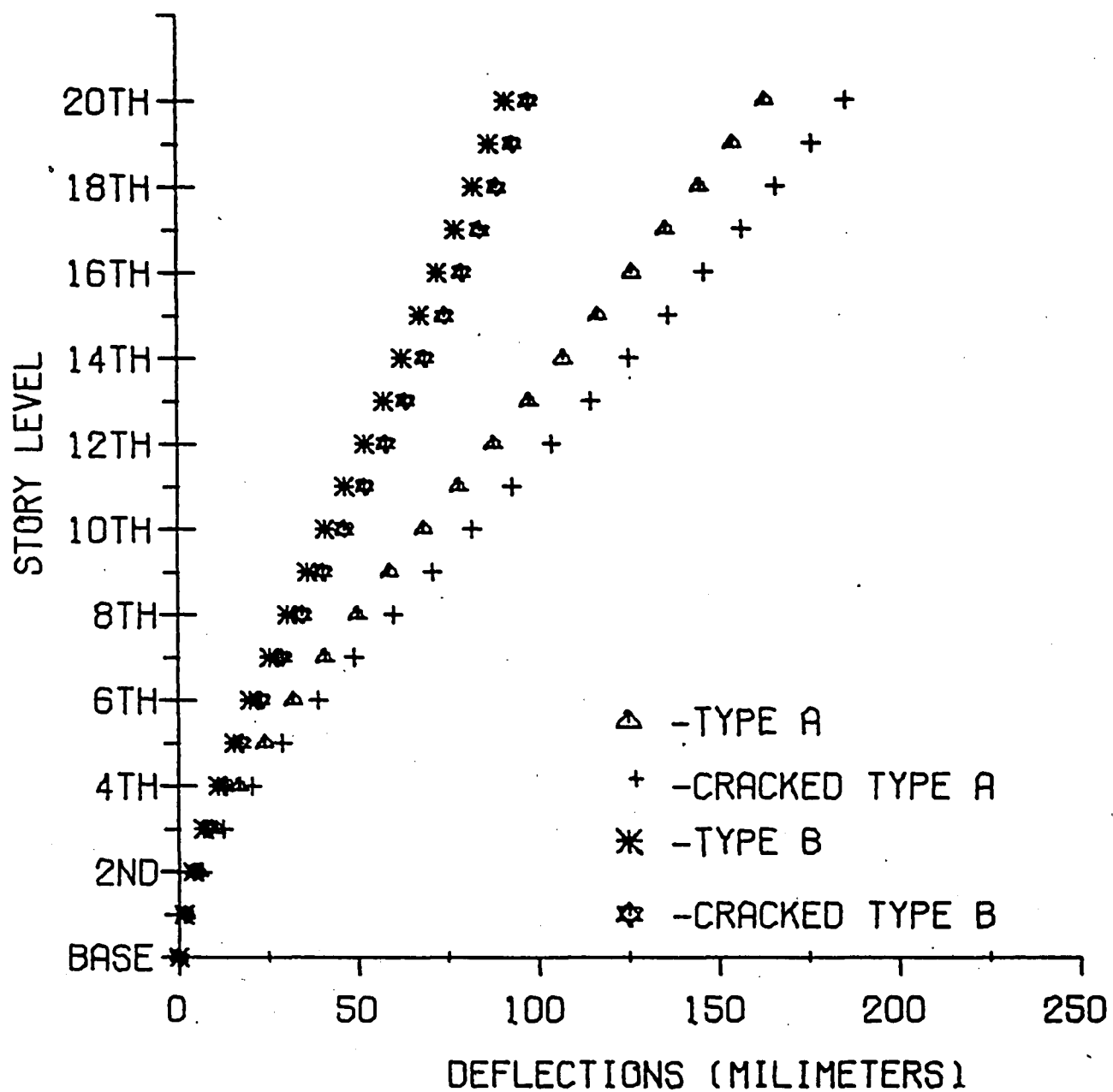
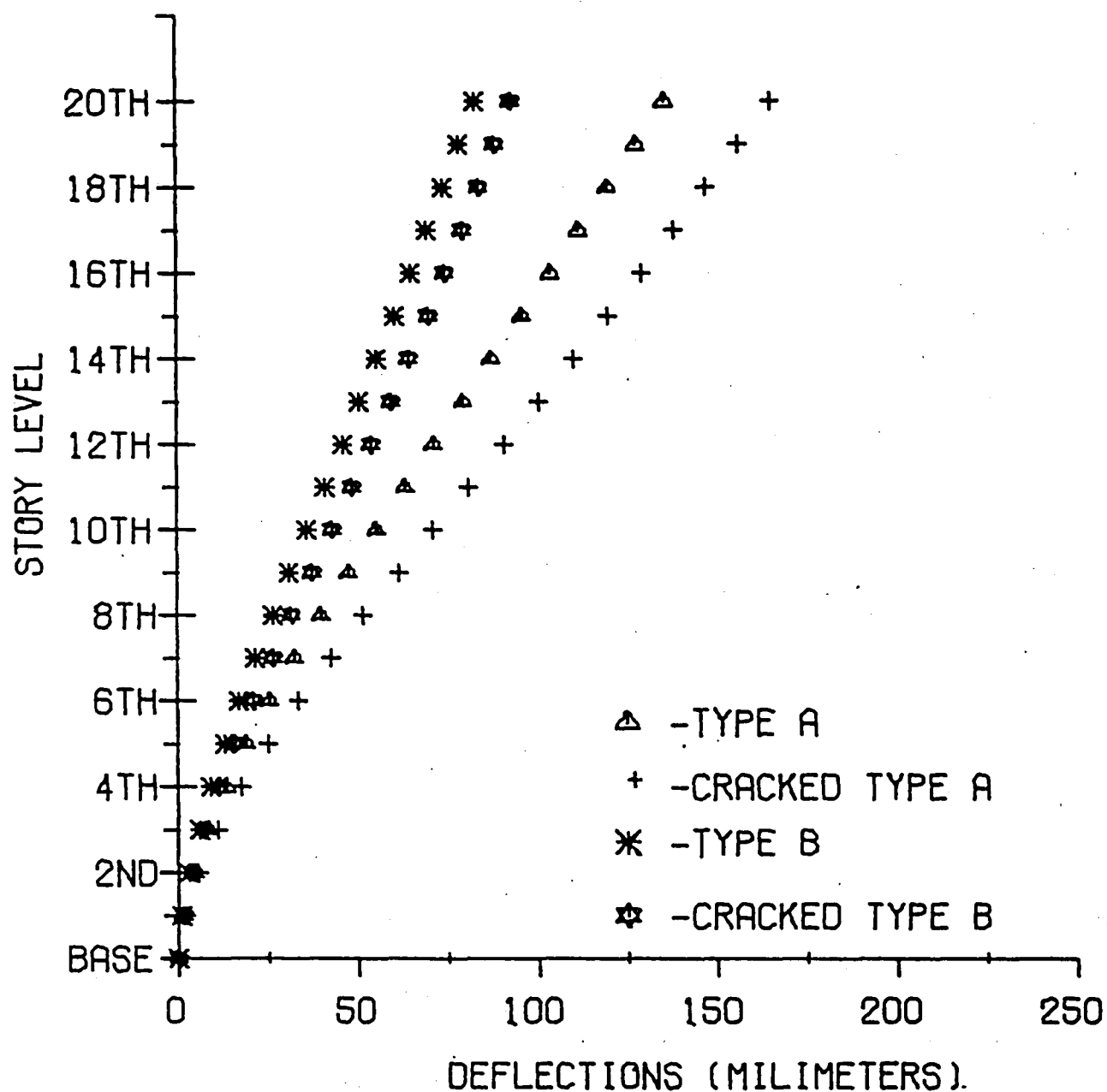


FIGURE 31

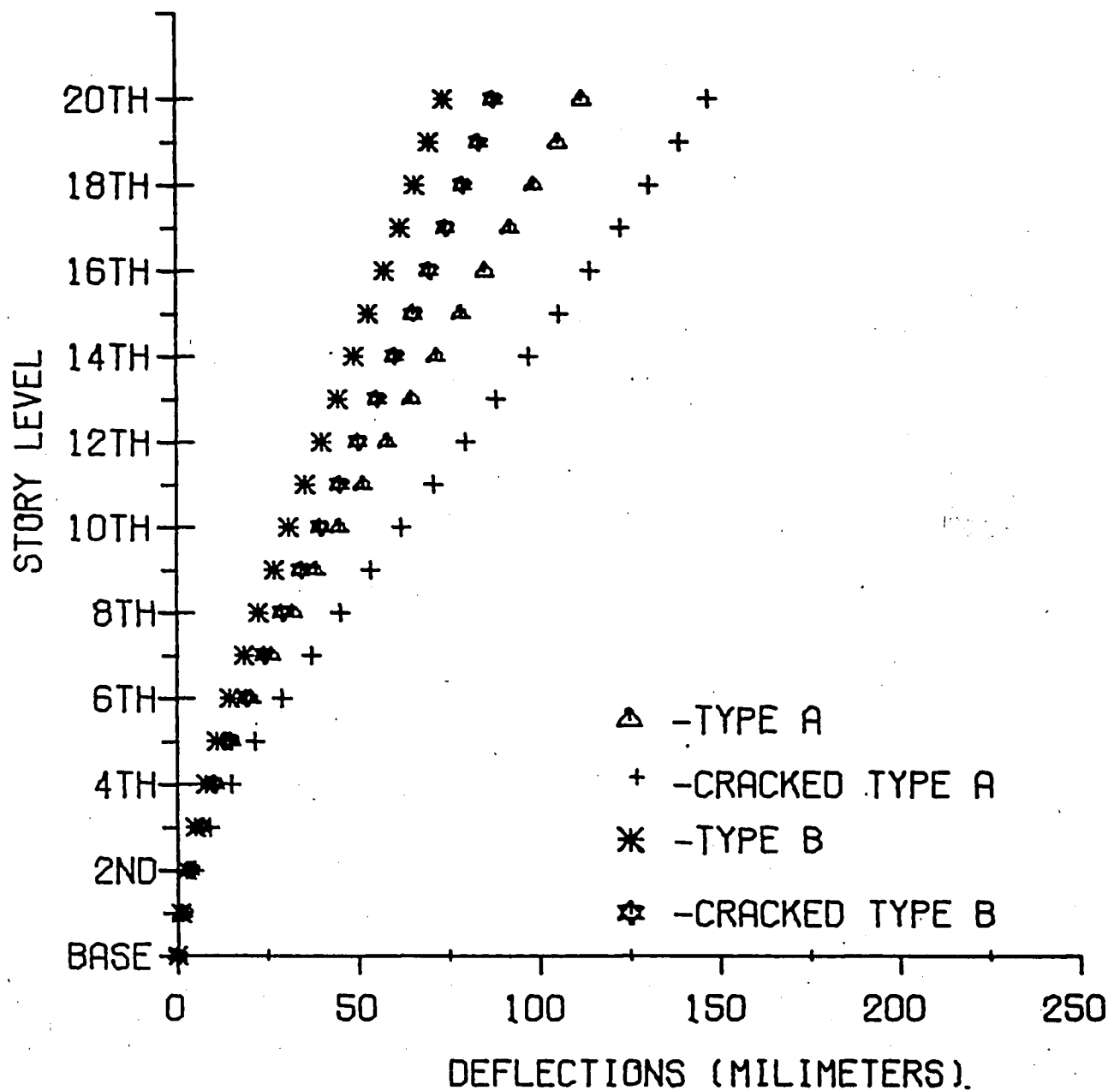


DEFLECTION PROFILES

FRAME 2 - CRACKED S. W. C

$$E_s = 0.25E_c$$

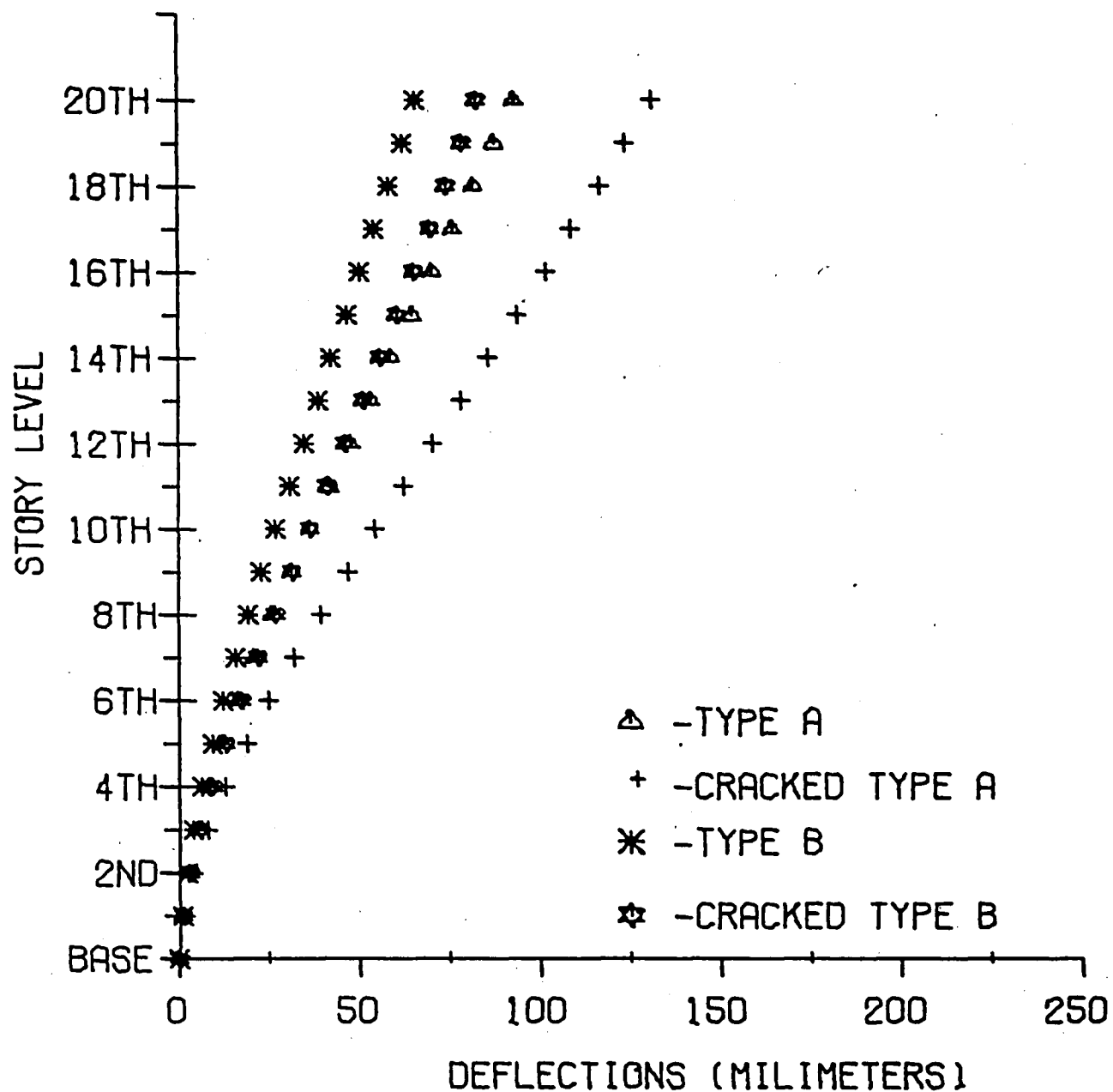
FIGURE 32



DEFLECTION PROFILES
 FRAME 2 - CRACKED S. W. D

$$E_s = 0.25E_c$$

FIGURE 33

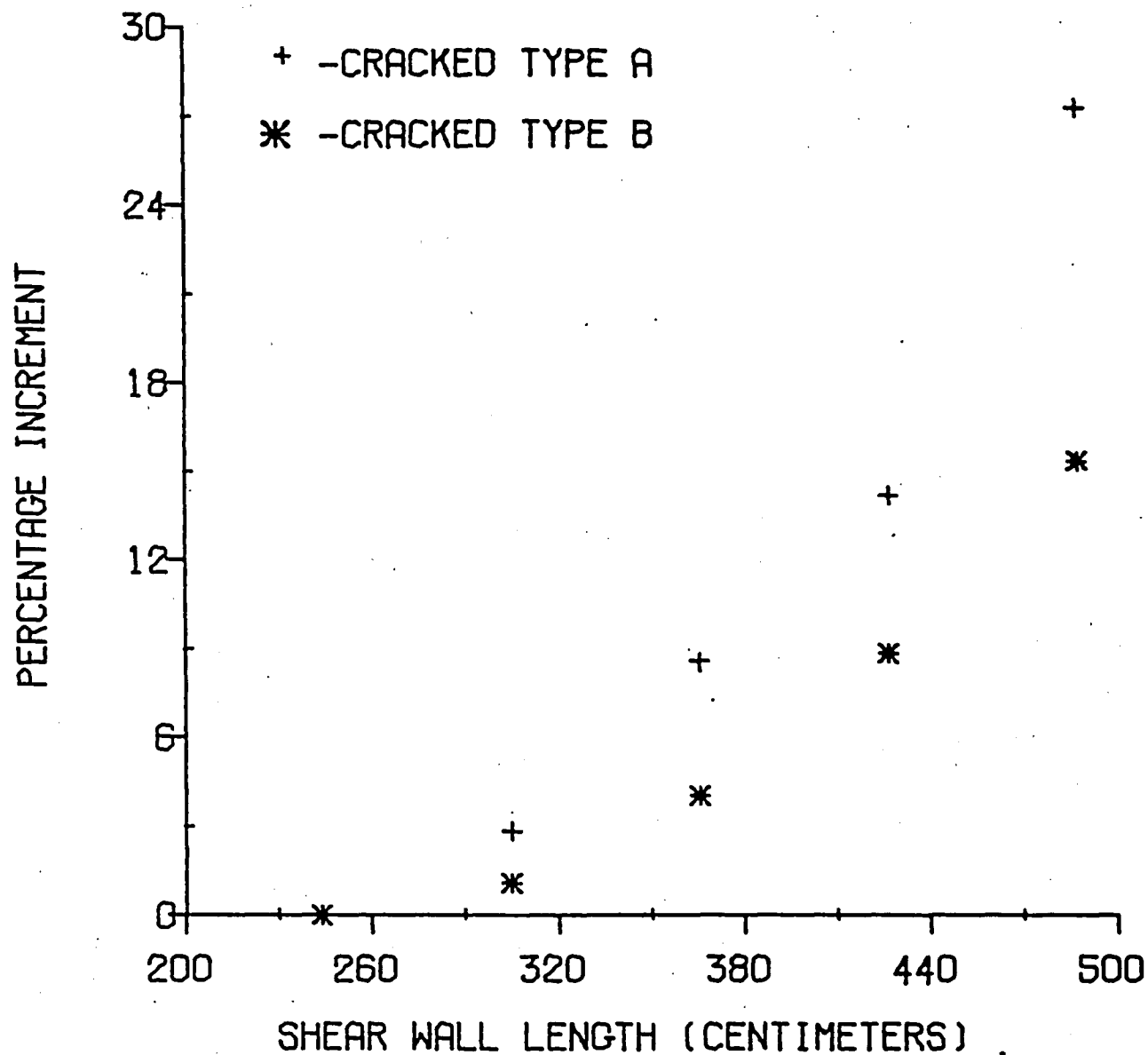


DEFLECTION PROFILES

FRAME 2 - CRACKED S. W. E

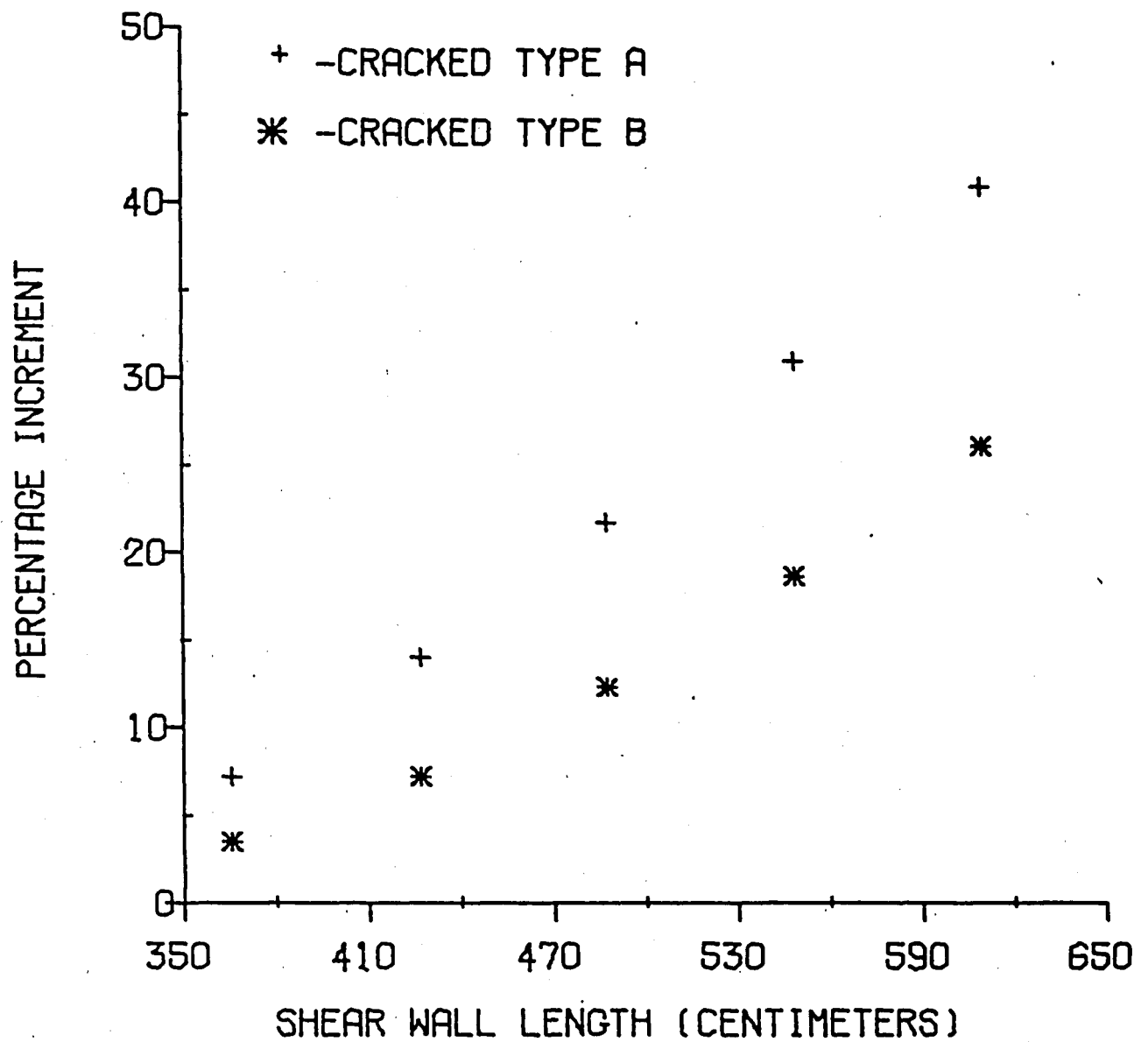
$$E_s = 0.25E_c$$

FIGURE 34



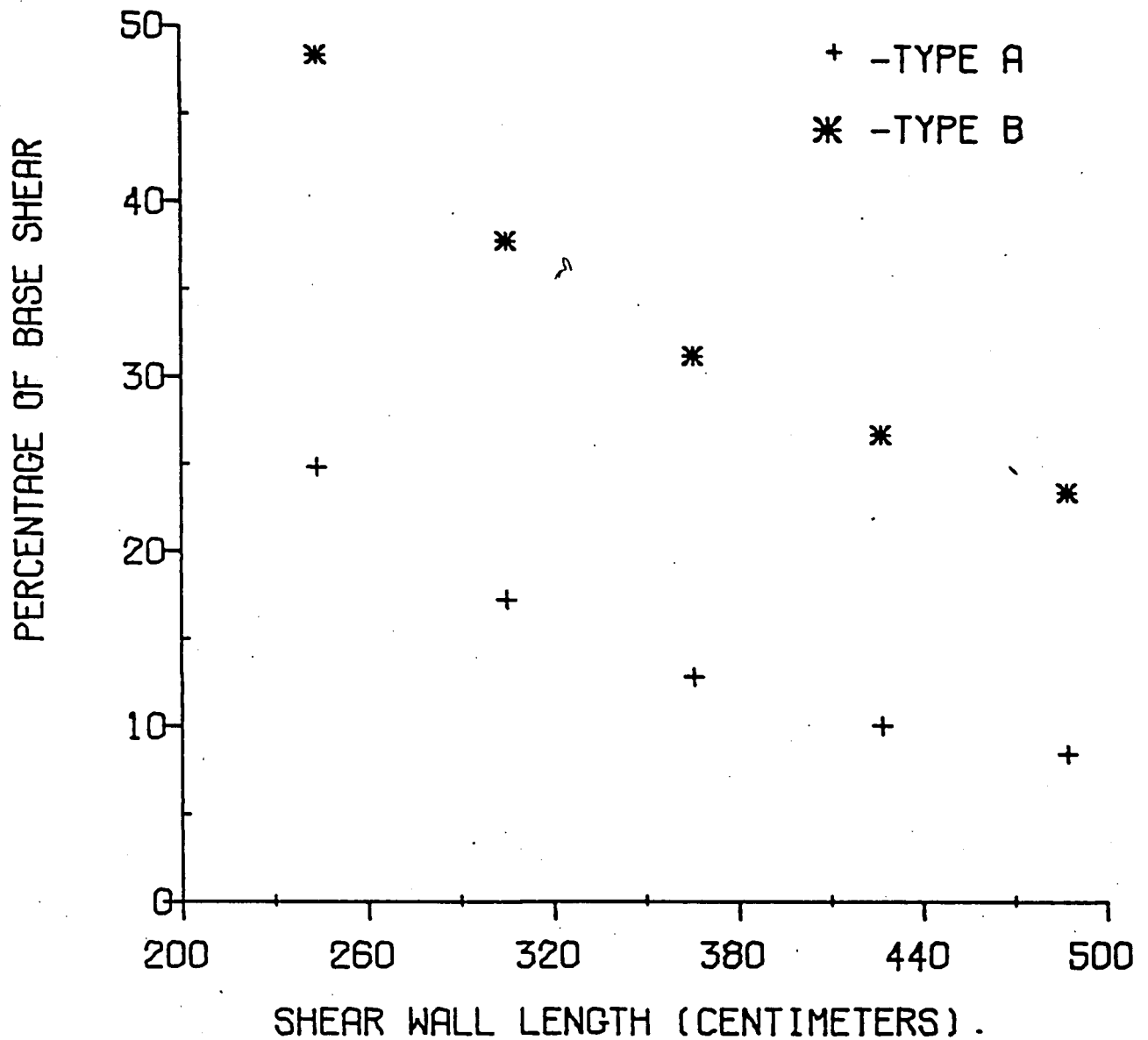
PERCENTAGE TOP DEFLECTION
INCREMENT FOR FRAME 1 AND
CRACKED WALL COMBINATIONS

FIGURE 35



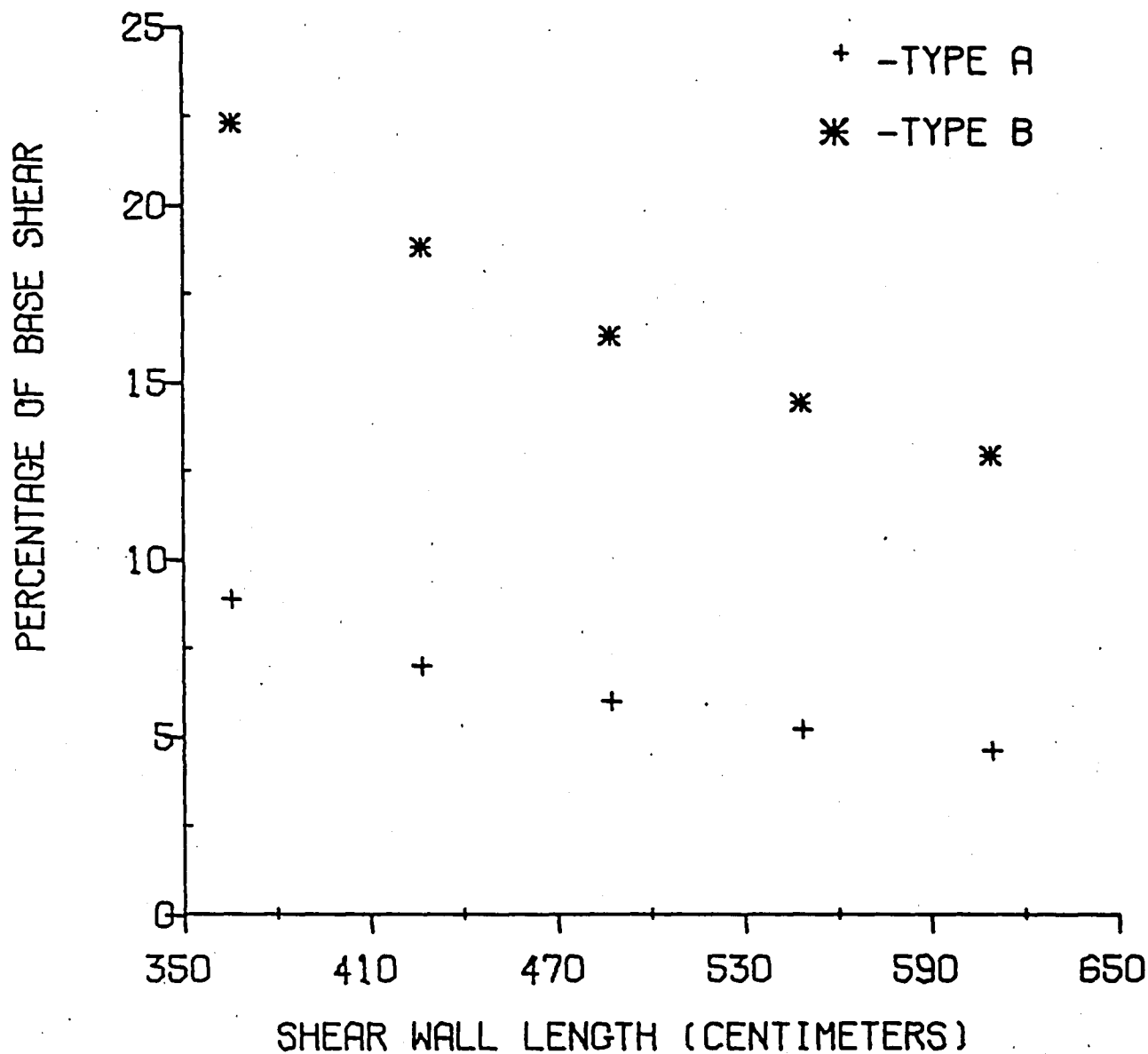
PERCENTAGE TOP DEFLECTION
INCREMENT FOR FRAME 2 AND
CRACKED WALL COMBINATIONS

FIGURE 36



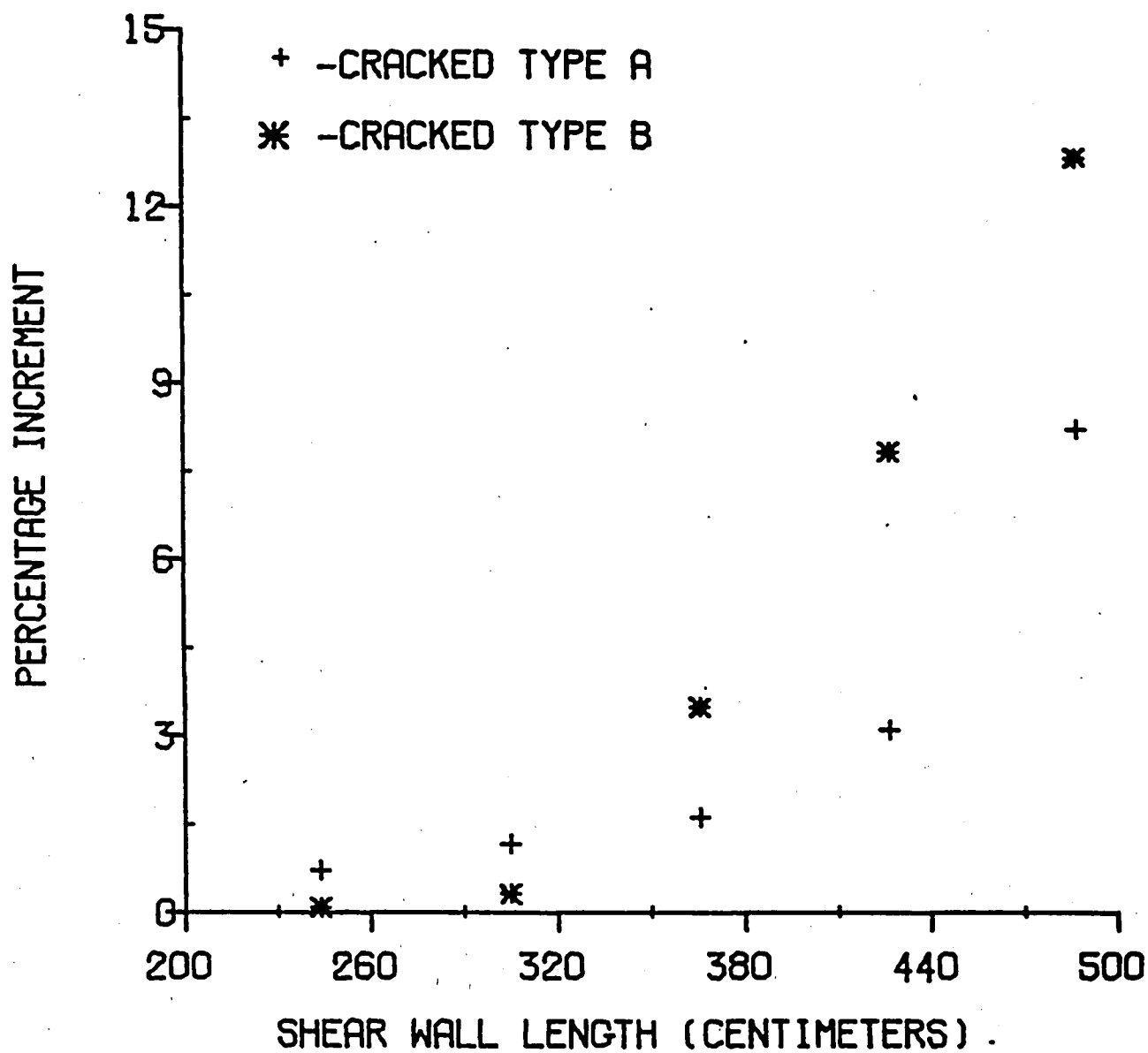
PERCENTAGES OF BASE SHEAR
ON FRAME 1 FOR CHOSEN
DIMENSIONS OF CRACKED WALL

FIGURE 37



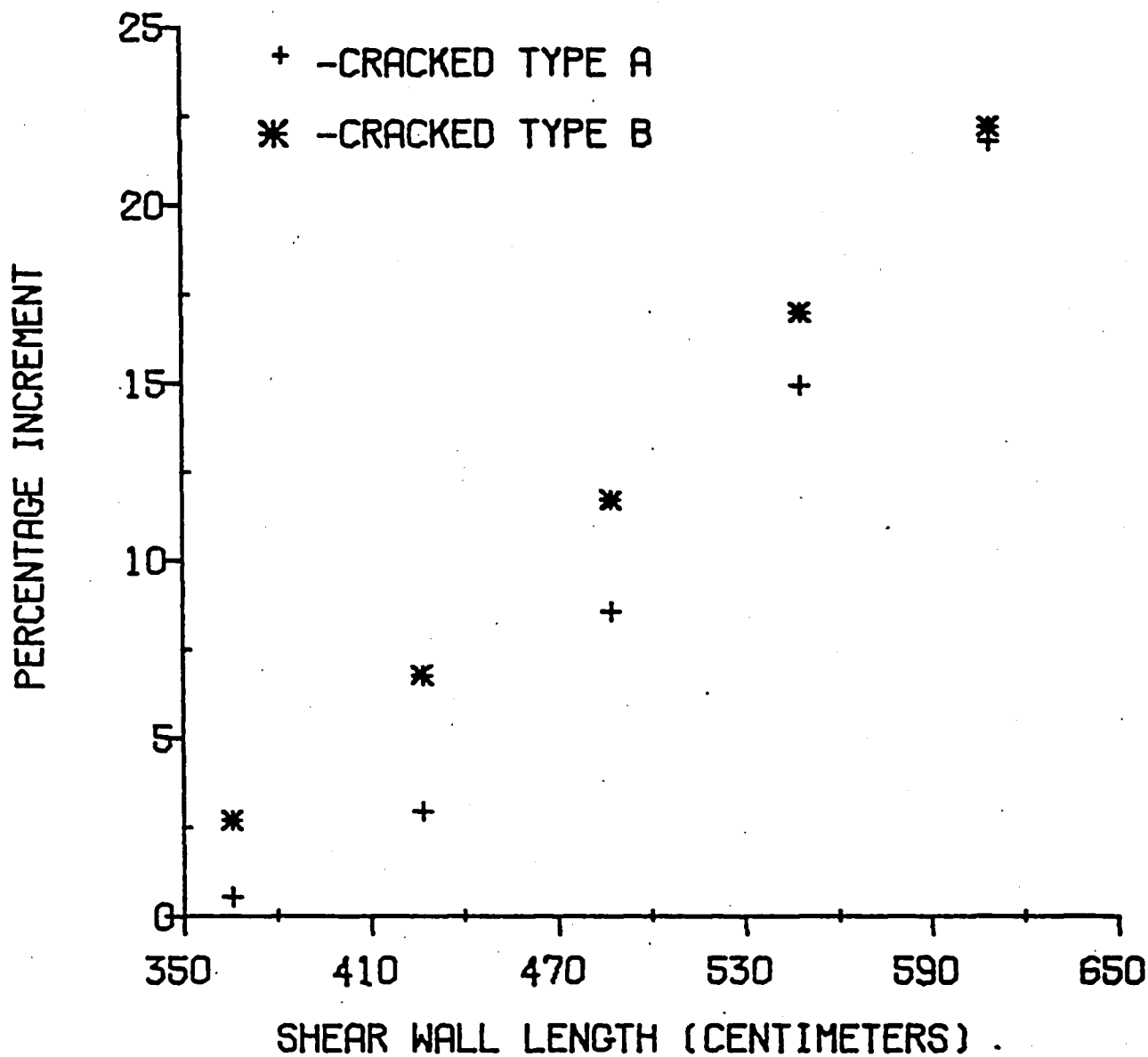
PERCENTAGES OF BASE SHEAR
ON FRAME 2 FOR CHOSEN
DIMENSIONS OF CRACKED WALL

FIGURE 38



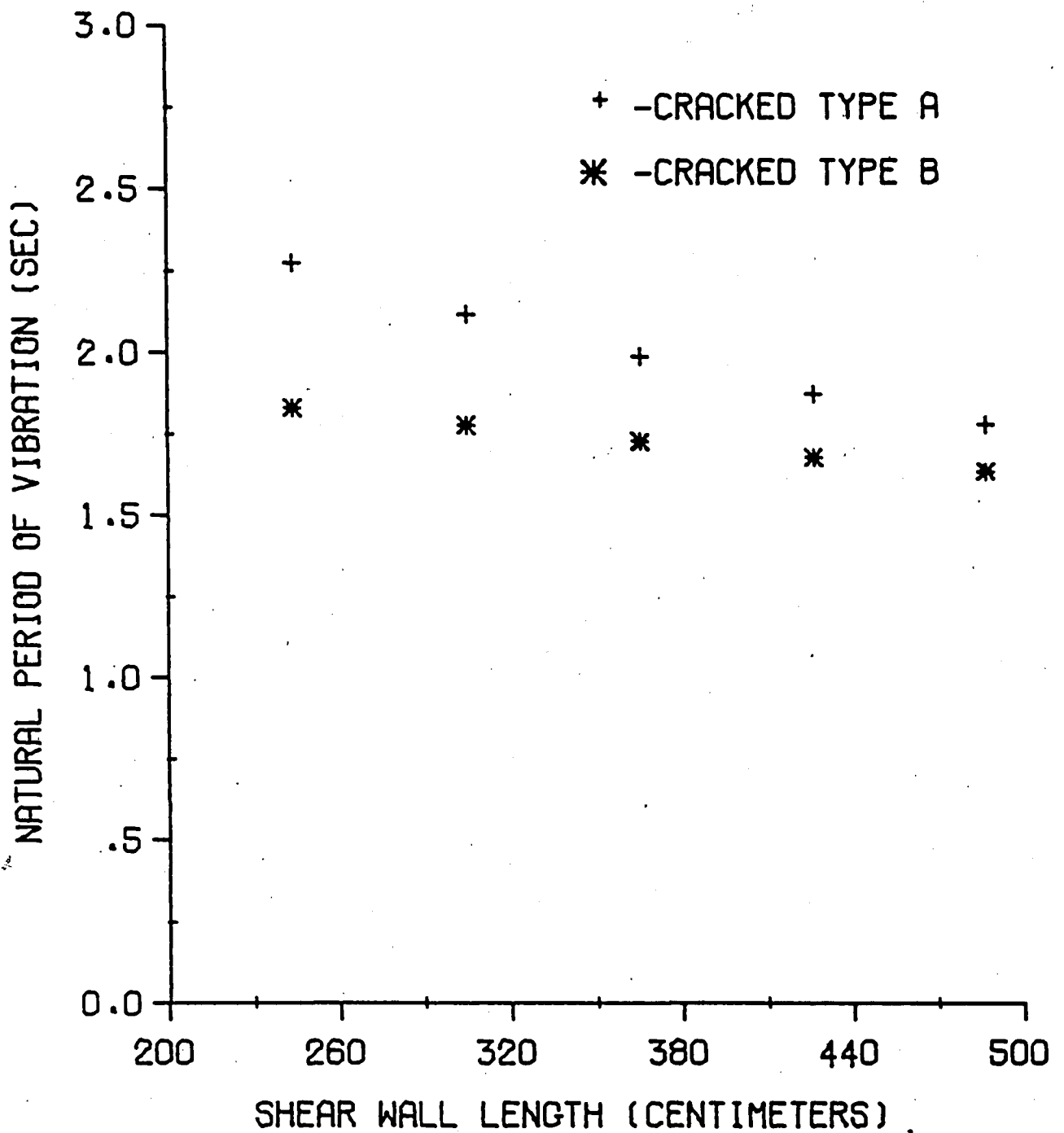
PERCENTAGE BASE SHEAR
INCREMENT FOR FRAME 1 -
CRACKED WALL COMBINATIONS

FIGURE 39



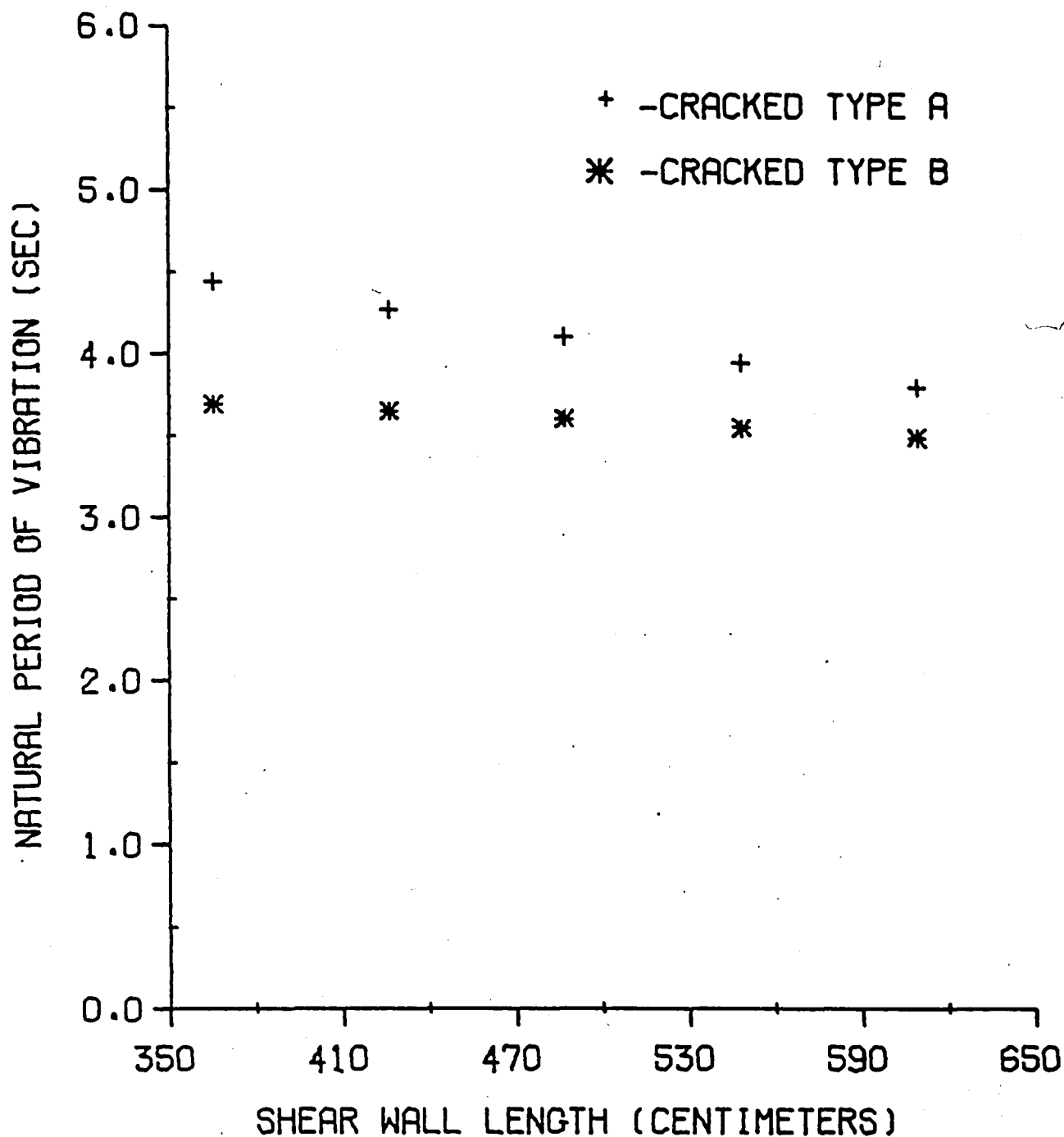
PERCENTAGE BASE SHEAR
 INCREMENT FOR FRAME 2 -
 CRACKED WALL COMBINATIONS

FIGURE 40



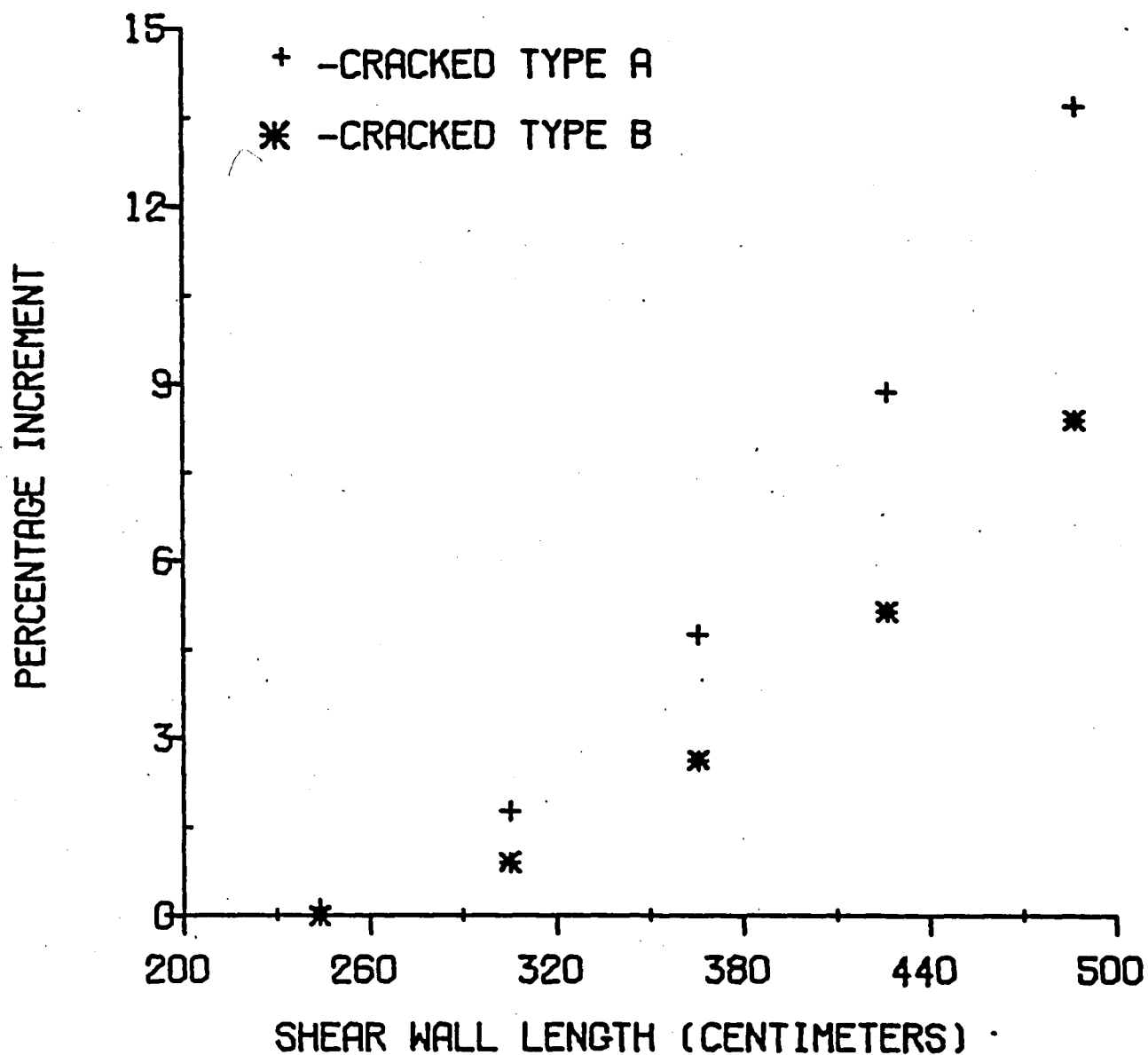
NATURAL PERIODS OF VIBRATION
FOR CHOSEN COMBINATIONS OF
FRAME 1 AND CRACKED WALLS

FIGURE 41



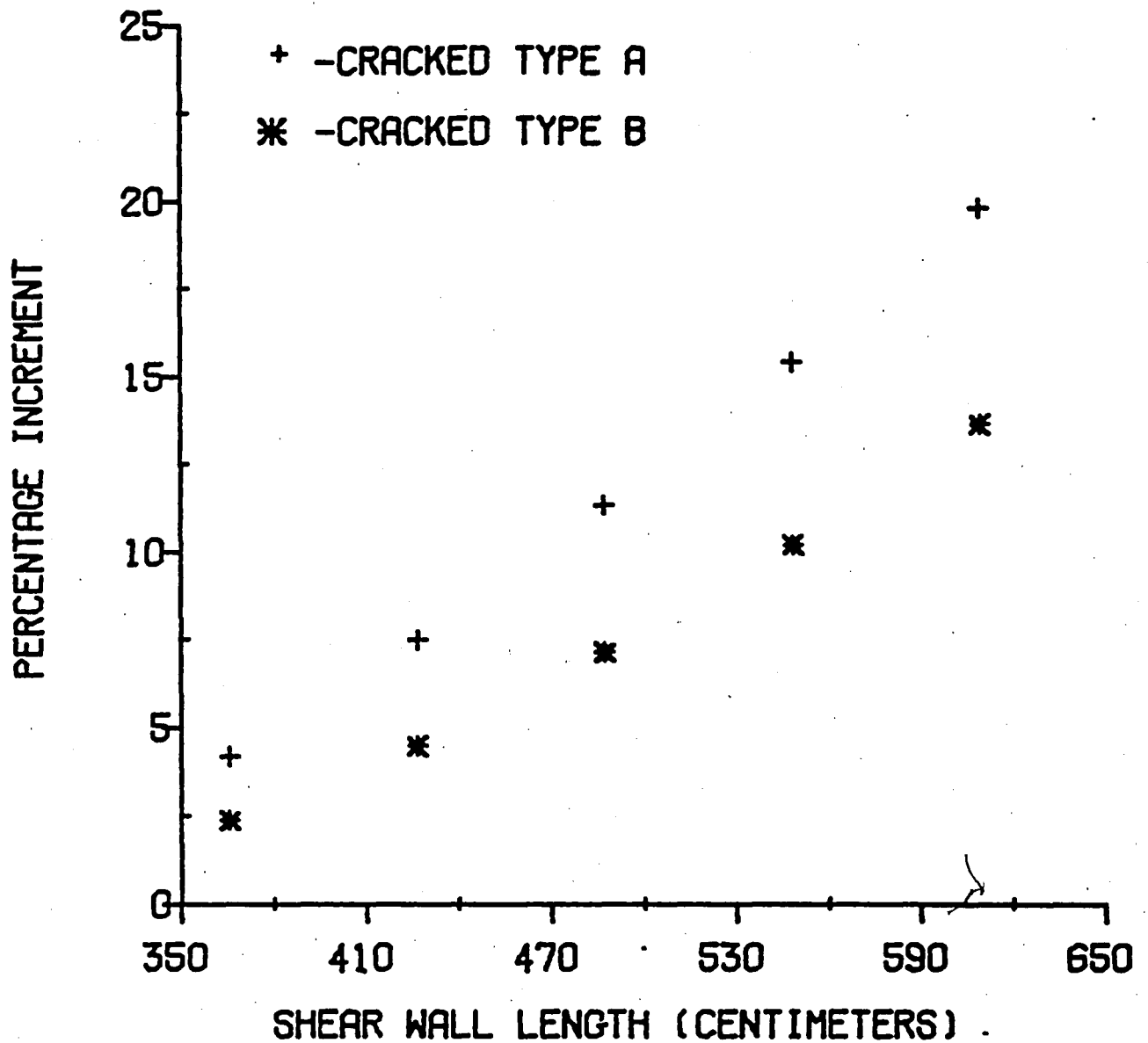
NATURAL PERIODS OF VIBRATION
FOR CHOSEN COMBINATIONS OF
FRAME 2 AND CRACKED WALLS

FIGURE 42



PERCENTAGE NATURAL PERIOD
INCREMENT FOR FRAME 1 -
CRACKED WALL COMBINATIONS

FIGURE 43



PERCENTAGE NATURAL PERIOD
INCREMENT FOR FRAME 2 -
CRACKED WALL COMBINATIONS

FIGURE 44

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APPENDIX A

APPROXIMATION OF FUNDAMENTAL PERIODS OF VIBRATION

As has been noted in the comparison of the natural periods of vibration of the reinforced concrete frame-shear wall combinations obtained via finite element analysis and Uniform Building Code provisions, discrepancies were noted (Ref. 20). Recent studies carried out by the Applied Technology Council have resulted in slightly different formula, found in the Commentary of the Provisions based on the results obtained in the San Fernando Earthquake field recordings (Ref. 19). For shear walled structural systems the traditional, e.g. UBC (Ref. 20), formula is

$$T_R = \frac{0.05 h_n}{\sqrt{D}} \quad (\text{Eq. A.1})$$

where h_n = the total building height (in feet) and

D = the "overall length" (in feet) of the building at the base in the direction under consideration.

The formula based on the field observations is similar, but with a slightly different coefficient:

$$T_R = \frac{0.07 h_n}{\sqrt{D}} \quad (\text{Eq. A.2})$$

The periods obtained by the former formula will be approximately 30% less than those obtained by the latter formula.

However, one of the major difficulties, or more precisely, the confusion, amongst the practicing engineers has been the

definition of the value "D" in the implementation of the formula. For example, for Frame 2, Type B configuration, if the shear wall length is 6.10 m, the value to be used by the practicing engineer can vary from $D = 6.10$ m (shear wall only) to $D = 24.10$ m (overall length of the building). The effects of choosing the "right-or-wrong" dimension are illustrated in Table A1.

In the establishment of Table A1, both Frame 1 and Frame 2, with their appropriate shear walls, are considered. In the table T_{FEM} corresponds to the period computed by the computer based finite element analysis. Subheadings "(A)" and "(B)" indicate the type of frame-shear wall assembly, which was previously described. The approximate periods are computed using Eq. A.2. Depending upon the choice of the length, D, three periods are computed. T_x corresponds to taking D as the length of the shear wall. T_y is arrived at by assuming that D is equal to the overall length of the building. This is similar to Type A frame-shear wall combination, in other words, the increase in the length due to the increase in shear wall length for connection Type B is not included. T_z corresponds to the full overall length of the building, which essentially simulates Type B arrangement.

Inspection of the periods indicates that T_y and T_z values are not close enough to any of the $T_{FEM} (A)$ or $T_{FEM} (B)$ values. Furthermore, because of the inherent small variations in the assumed lengths for T_y and T_z , the variations are extremely small, as expected. T_x always provides an upper bound to T_{FEM} .

The contents of Table A1 clearly indicate that further definitions, and improvements, are in order to develop a more reliable formula than those that are frequently used or tentatively proposed.

TABLE A1: FUNDAMENTAL PERIODS (IN SECONDS)

BY VARIOUS ASSUMPTIONS

	Shear Wall Length (cm)	T _{FEM}		T _x	T _y	T _z
		(A)	(B)			
Frame 1 and Shear Wall	243.84	2.27	1.83	3.02	0.95	0.91
	304.80	2.08	1.76	2.70	0.95	0.90
	365.76	1.89	1.68	2.47	0.95	0.89
	426.72	1.72	1.59	2.28	0.95	0.88
	487.68	1.56	1.51	2.14	0.95	0.87
Frame 2 and Shear Wall	365.76	4.26	3.61	4.85	2.17	1.98
	426.72	3.96	3.49	4.49	2.17	1.95
	487.68	3.67	3.35	4.20	2.17	1.93
	548.64	3.41	3.21	3.96	2.17	1.90
	609.60	3.16	3.06	3.76	2.17	1.88

VITA

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